rt No. FHWA-RD-78-23

EGRATED MOTORIST INFORMATION SYSTEM (IMIS) SIBILITY AND DESIGN STUDY

Phase II: Generalized Methodology for IMIS Feasibility Studies

Vol. 1 IMIS Feasibility Study Handbook

DEPARTMENT OF TRANSPORTATION ★ UNITED STATES OF AMERICA ★

May 1978

Final Report

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Prepared for

FEDERAL HIGHWAY ADMINISTRATION
Offices of Research & Development
Washington, D.C. 20590

FOREWORD

This handbook presents a methodology for conducting a feasibility study for an Integrated Motorist Information System (IMIS) in any freeway corridor. It also contains information on costs and tradeoff considerations for the selection of traffic surveillance and control subsystem elements and techniques. This handbook is intended for practicing traffic engineers, as a guide for performing an IMIS feasibility study.

A companion volume (Volume 2), "Validation and Application of Feasibility Study Handbook," FHWA-RD-78-24, presents the results of applying the handbook methodology to a test corridor in California and a validation of the benefit assessment methodology.

These two volumes constitute the Final Report on Phase II: Generalized Methodology for IMIS Feasibility Studies, which is the second of three phases of the "Integrated Motorist Information System Feasibility and Design Study," conducted for the Federal Highway Administration, Office of Research, Washington, D. C., by Sperry Systems Management under Contract DOT-FH-11-8871.

Phase I, a feasibility study for an IMIS in the Northern Long Island Corridor in New York State, was reported in three volumes: Final Report, FHWA-RD-77-47; Appendices, FHWA-RD-77-48; and Executive Summary, FHWA-RD-77-49. Phase III will result in the final design for an IMIS in the corridor studied in Phase I.

Sufficient copies of this report are being distributed by FHWA Bulletin to provide a minimum of one copy to each FHWA Regional Office, one copy to each FHWA Division Office, and one copy to each State highway agency. Direct distribution is being made to the Division offices.

A limited number of additional copies for official use are available upon request from the Systems Development and Technology Group, HRS-32, Traffic Systems Division, Office of Research, FHWA, Washington, D. C. 20590.

Charles F. Scheffey
Director, Office of Research

NOTICE

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16. Abstract

Phase I of the IMIS project examined the feasibility of implementing an Integrated Motorist Information System in a corridor located in New York (northern Long Island). The results are documented in report numbers FHWA-RD-77-47, 48 and 49. Under Phase II of IMIS (the present phase), the methodology for performing an IMIS feasibility study was generalized and put into "handbook" form to provide practicing traffic engineers with a step-by-step procedure for accomplishing such a study. Major topics included in the handbook are: criteria for IMIS Applicability, assembly of the necessary data base, selection and evaluation of candidate routes, equipment trade-offs, development of alternative preliminary designs, and system evaluation on a benefit/cost basis.

Although the handbook is not intended as a detailed design guide, it addresses subjects requiring design related decisions, such as selection of communications media, alternate routes, diversion points, and ramp metering locations. As such, it should be of general interest to those concerned with freeway surveillance and control system design.

A second volume, identified below, presents the results of applying the handbook to a test corridor. It contains a complete example of an IMIS feasibility study and should therefore be of particular interest to handbook users.

This volume is the first in a series. The other volume in the series is:

Vol No.	FHWA No.	Short Title	NTIS (PB) No.
2	78-24	Feasibility Handbook Application	(not yet available)

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CHAPTER 1

INTRODUCTION

As traffic demands continue to increase, and economic and social pressures continue to preclude any further significant major roadway construction (particularly in urbanized areas), the need for more efficent utilization of existing highway networks has become paramount. Recognition and response to this situation is evidenced by the investments which have been and continue to be made by the Federal Government in support of research and operational improvements for these facilities.

The underlying premise of the Integrated Motorist Information System (IMIS) concept is that present state-of-the-art equipment and real-time traffic responsive control techniques can be used for effective management of traffic in an existing highway network, or "corridor". Further, by combining individual remedial measures in an integrated design, a more cost-effective system, capable of addressing a greater portion of the traffic-related problems, can be provided.

The "backbone" of IMIS is an electronic surveillance subsystem, which provides the real-time data required to monitor and control traffic in the corridor. The electronic surveillance also provides an automatic incident detection capability so that the system can react to problems in the shortest possible time. Other forms of manual surveillance may be incorporated, or integrated if existing, to improve system response.

The major control techniques used in IMIS include route diversion, ramp metering, and arterial signal control. Often, these controls will operate in conjunction with each other. For example, consider a "corridor" which includes two parallel freeways and a nearby parallel arterial or service road. If a problem develops on one freeway, some of its traffic could be diverted to the other freeway. If there is insufficient spare capacity on the latter, upstream ramp metering can be used to reduce or limit its oncoming volume. This could induce some secondary diversion of the ramp traffic to the arterial or service road. There, computer control of the signals would be used to increase capacity, so as to accommodate the additional traffic with minimal impact. Numerous combinations of these control techniques can be applied in a corridor, depending on prevailing conditions. It is significant to note that such a capability greatly expands the number of opportunities to alleviate problems, and can thus maximize the utilization of the roadway network.

Coupled with the surveillance and control functions, IMIS provides a comprehensive system of motorist information and services. These include:

- Variable message signs - located at key locations throughout the corridor to provide real-time traffic advisory, alternate routing, and other pertinent information to the motorist.

- Highway advisory radio - consisting of roadside transmitters providing traffic and related information, receivable at certain frequencies on the standard AM radio.

- Motorist air/emergency - roadside call boxes along the freeways for contacting cognizant response agencies to assist disabled motorists.

- Trip information services - various forms to aid motorists in pretrip and enroute planning.

Implementation of IMIS, with features as noted previously, involves a substantial capital investment. Furthermore, there must be a concomitant investment in, and commitment to, maintaining the continued operational capability of the system - otherwise, even the best technically designed system will be of little value.

Is the total investment in IMIS warranted for a given corridor? The objective of this handbook is to assist the using agency in answering this question. To achieve this objective, the handbook provides a framework and methodology for performing an IMIS feasibility study for any corridor. The feasibility study includes the development of a series of alternative system designs and their subsequent evaluation on a benefit/cost ratio basis. The designs are preliminary in nature, and are developed only to the extent necessary to provide a credible benefit/cost evaluation. The designs are, however, geared toward providing a spectrum of system roadway networks and costs to allow a choice to be made consistent with desired functional capability and funds available. If the feasibility study results are positive the alternative designs will provide guidance for the selection of a system for implementation. The selected design need not be identical in every aspect to one of the candidates, i.e., it can be modified to suit specific needs or budget constraints. Any significant modification, however, should be assessed to determine whether a reevaluation of the benefit/cost ratio is considered necessary.

A substantial effort is normally involved in developing system software. In recognition of this, the Federal Highway Administration has sponsored the development of a generalized control algorithm which will optimize traffic flow in any corridor (Contact No. DOT-FH-11-8738, "Development of Traffic Logic for Optimizing Traffic Flow in an Intercity Corridor"). Thus, the most complex portion of the overall system software will be available for use in IMIS.

As noted earlier, the purpose of IMIS is to improve the utilization of the existing highway network. As such, it does not include any new major roadway construction as part of the IMIS design per se. However, any planned construction is considered in the IMIS study to the extent that it affects the roadway configuration and traffic operations.

Similarly, IMIS should not be considered as a "catch all" for all needed improvements in the area. For example, if an isolated arterial in the corridor has recurrent problems, but is not a suitable alternative route for the IMIS network, it is

not considered as part of IMIS. Remedial measures for that arterial should be considered in a separate study.

The organization of the handbook is described in Chapter 2. Subsequent sections provide the sequential procedure for performing the feasibility study. To the extent possible, typical examples*, guidelines and/or recommendations are provided to assist the user in accomplishing the study. However, the single most valuable resource is the user's experience and familiarity with his own highway network and its operations. Full use should be made of this resource throughout the study.

It is noted that a second report for this project (referenced on the documentation page) contains an application of the handbook to a "test corridor" in California. As such, it should be of particular interest to the handbook user in that it provides a complete example of a feasibility study based on the handbook methodology. Finally, both the handbook and application report may be useful as a reference for various system design and evaluation aspects, even if a total IMIS feasibility study is not being considered.

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*For the most part, the typical examples provided were taken from the IMIS Feasibility Study performed for the northern Long Island corridor in New York, the results of which are contained in the following documents: "Integrated Motorist Information System (IMIS) Feasibility and Design Study, Phase I: Feasibility Study," "Report Nos. FHWA-RD-77-47 (Final Report) and FHWS-RD-77-48 (Appendixes), Federal Highway Administration, Washington, D.C., April 1977.

CHAPTER 2

HANDBOOK ORGANIZATION

The IMIS feasibility handbook provides the practicing traffic engineer with a step-by-step procedure for performing a feasibility study of an Integrated Motorist Information System. The organization of the handbook follows the fourteen sequential tasks given in Figure 1. Each chapter of the handbook discusses one of these tasks. In this way, a convenient format for guiding the user through the methodology is obtained. An important characteristics of the handbook structure is the single path provided for completion of the tasks. The user is thus let through the methodology with the understanding that each task is to be completed before the next is started.

The Introduction section of each chapter has been put into the following standardized format:

(1) objectives of task
(2) inputs required by task
(3) outputs of task

This information provides an orientation for the user to assist in understanding and completing the task. The remaining sections of each chapter provide the necessary background material and procedures for completing the task.

The handbook is a step-by-step evaluation and selection process, but not a complete reference book on corridor systems. It must be used with general references (such as the Traffic Control Systems Handbook and the Highway Capacity Manual) available to traffic engineers.

Although the handbook is not intended as a detailed design guide, the methodology does, however, move into areas requiring design related decisions, especially in the area of communication, roadway network, and diversion and ramp metering locations. These system configuration decisions are critical for developing the system costs and benefits. For developing candidate designs, there is no established set of tradeoff relationships. The alternative systems are developed from numerous combinations of system components and corridor facilities. Thus, the methodology is structured to lead the user through the design related decisions in developing alternatives, assigning costs and assessing accrued benefits.

As noted, the feasibility study will lead to a series of alternative designs and their evaluation on a benefit/cost basis. While the handbook has been structured to minimize the user effort to reach this point, a credible study could nevertheless require a substantial amount of work on the user's part. (This will depend to a large extent on the available data base). To preclude expending this effort for a corridor which in fact is not suitable for an IMIS treatment, the next chapter contains a preliminary study to determine "IMIS applicability". If the results of this

assessment are negative, the indication is that system justification is unlikely on a benefit/cost basis. The user may then elect to terminate the remainder of the feasibility study, particularly if his resources are limited.

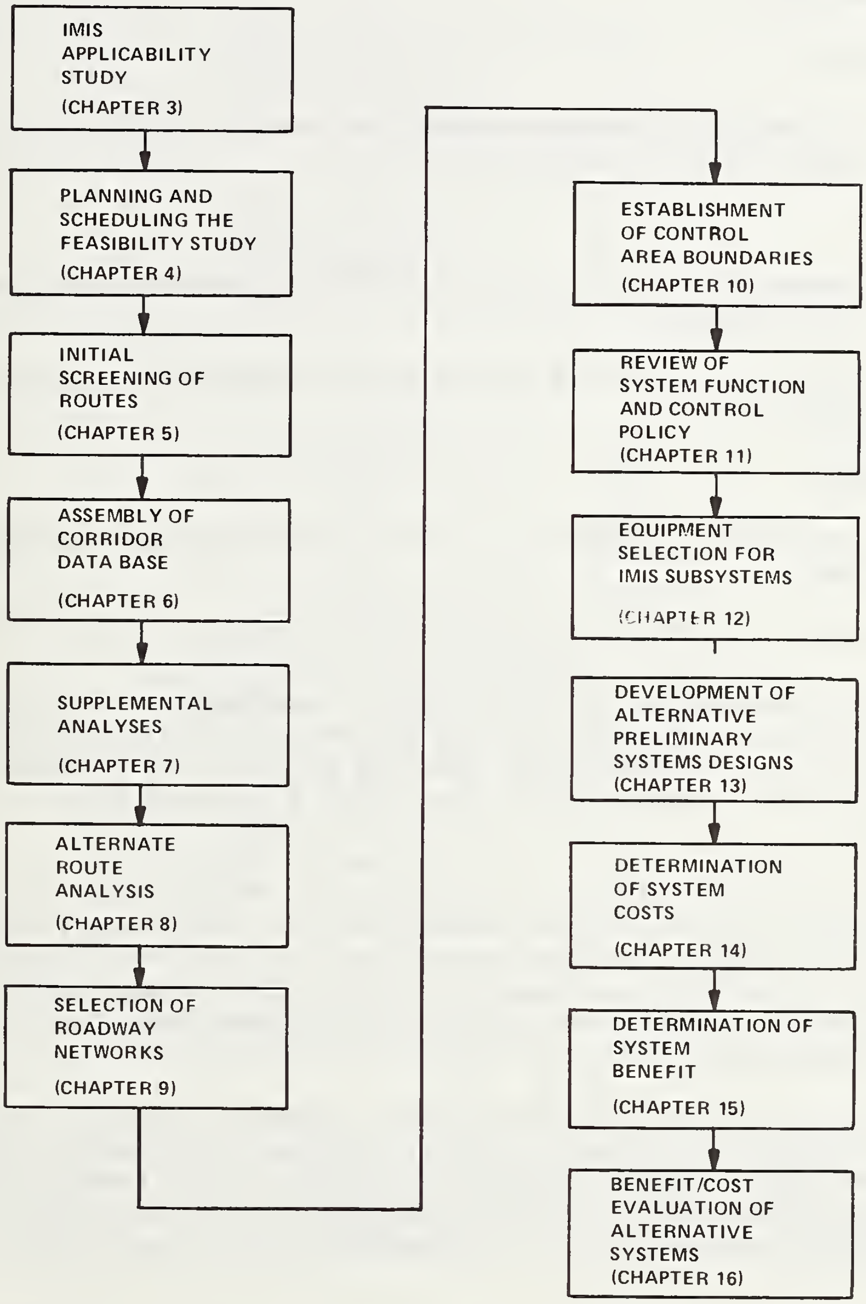


Figure 1. IMIS Feasibility Study Task Sequence

CHAPTER 3

IMIS APPLICABILITY STUDY

3.1 INTRODUCTION

3.1.1 Objectives

- To determine whether to proceed with the feasibility and preliminary design study for the given corridor.

3.1.2 Inputs

- General knowledge of corridor roadways including general operational characteristics.
- Average speed data for limited access facilities during peak demand periods.

3.1.3 Outputs

- A decision to proceed with or stop the feasibility and preliminary design study.

3.2 OVERVIEW OF APPLICABILITY STUDY PROCEDURE

The objective of the applicability study is to determine fairly quickly, using simplified procedures, whether a corridor has any potential for a cost-effective IMIS. If the result of the applicability study are positive, then the agency enters into a more detailed feasibility study. If not, then it has saved the cost of the more detailed study.

The applicability study proceeds according to the flow chart illustrated in Figure 2. First, the basic corridor is identified, i.e., the primary routes, the potential alternate routes, and appropriate corridor boundaries. Then, general guidelines are reviewed to determine if the basic elements are present for successful corridor operation. If the corridor generally falls within these guidelines, then the remaining applicability study is conducted.

These guidelines should not be viewed as hard and fast rules which cannot be violated. Rather they indicate the kinds of corridors most likely to result in a cost-effective Integrated Motorist Information System. Since there undoubtedly will be atypical situations where an IMIS would also be of value, the judgment of the traffic engineer is still critical in making the decision to proceed with the feasibility study.

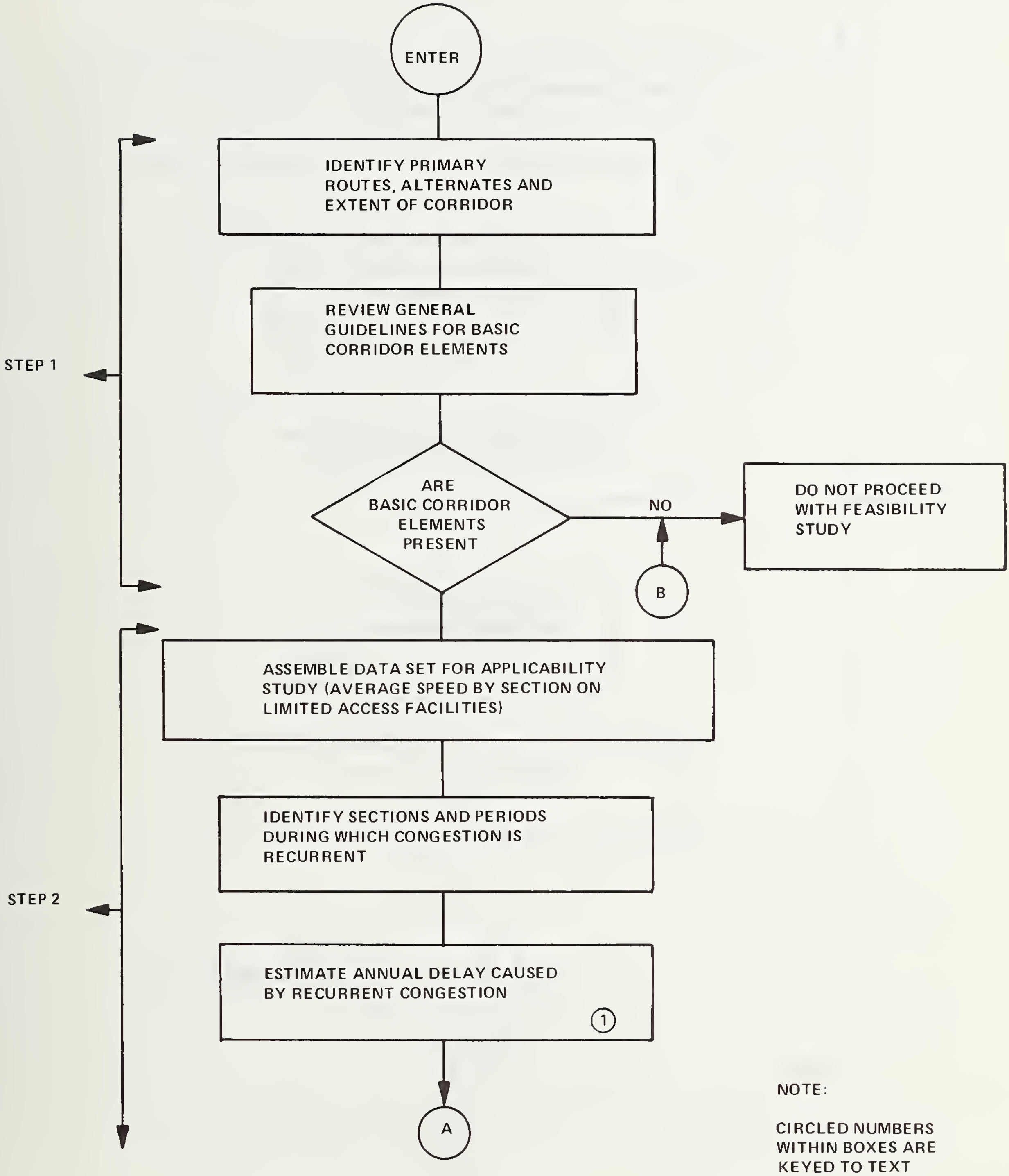


Figure 2. IMIS Applicability Study Flow Chart (Sheet 1 of 3)

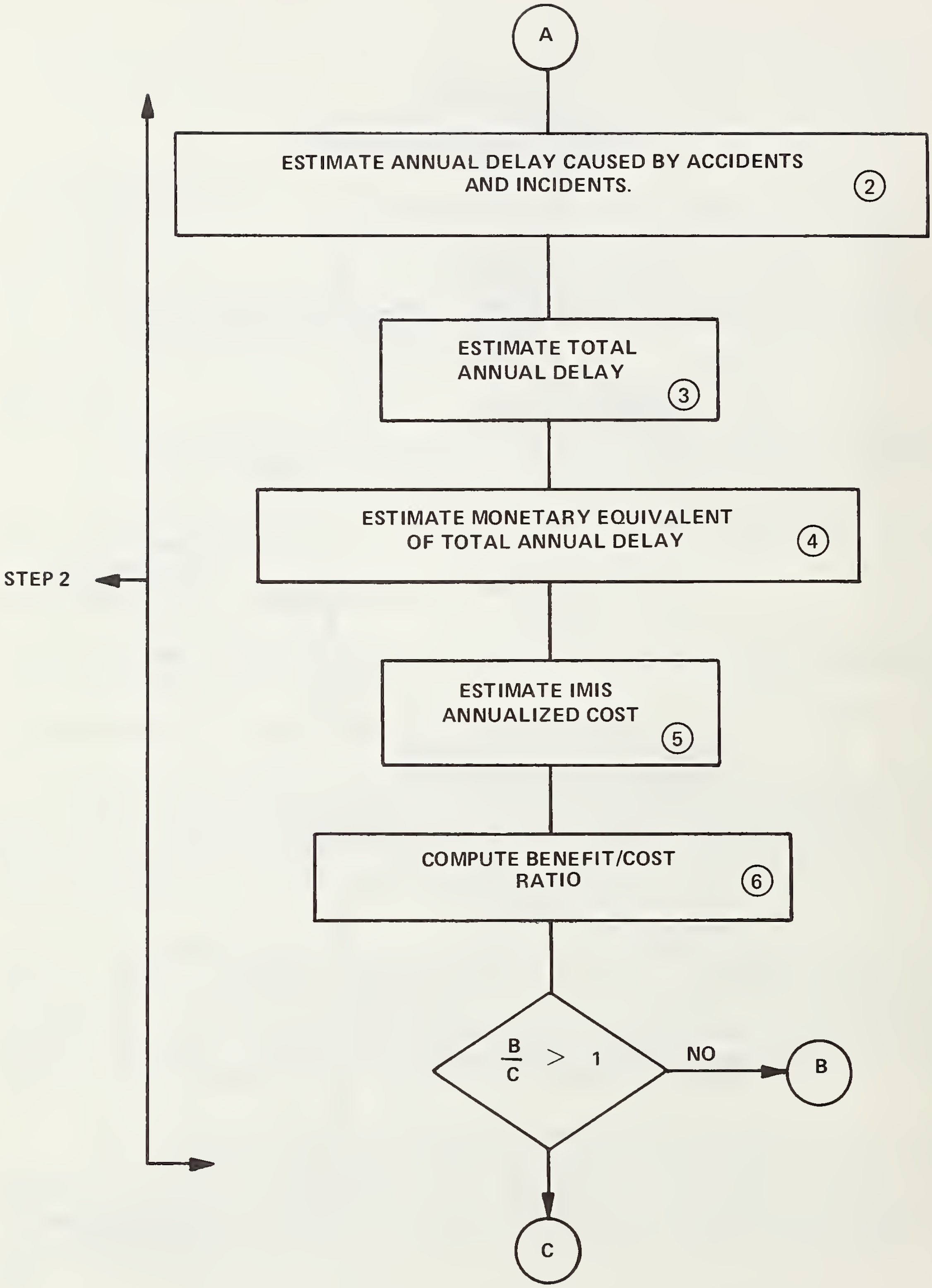


Figure 2. IMIS Applicability Study Flow Chart (Continued)

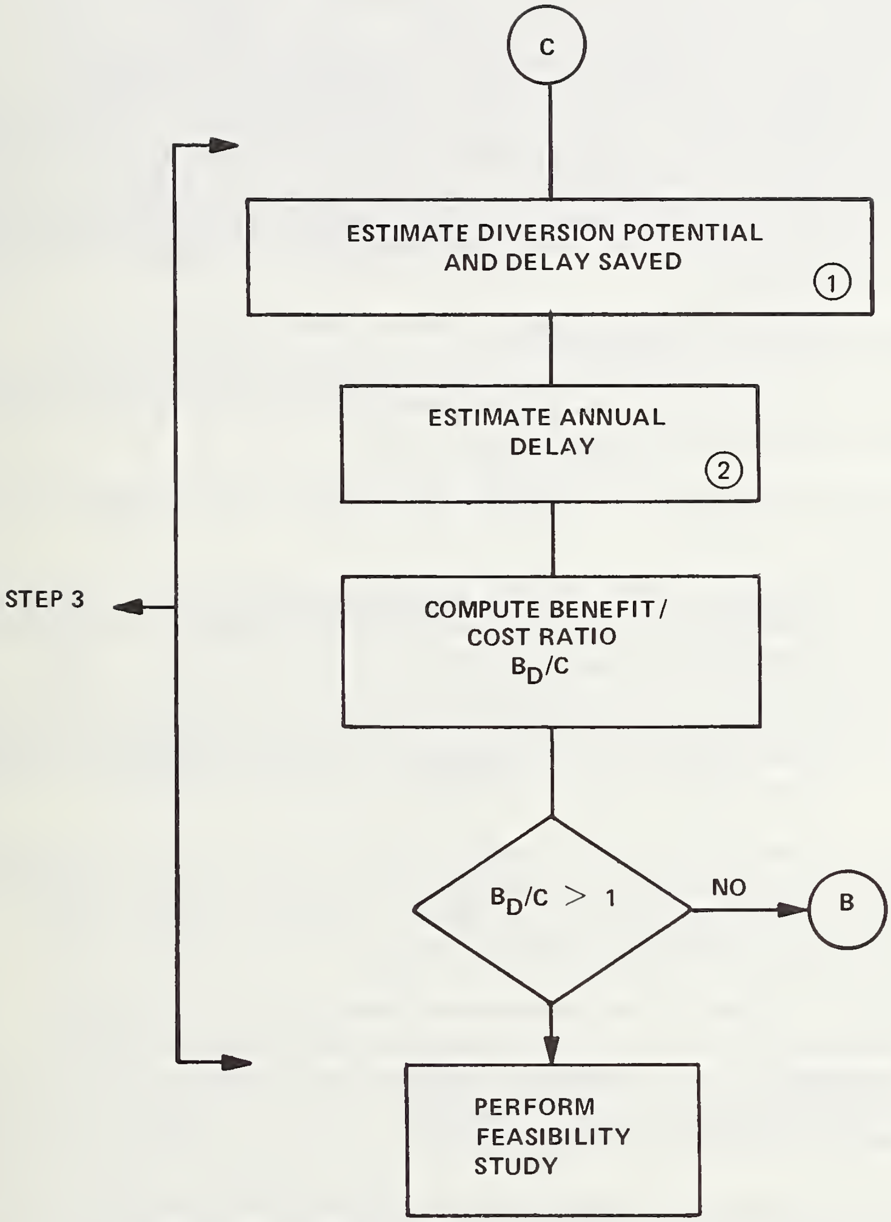


Figure 2. IMIS Applicability Study Flow Chart (Continued)

The necessary data for the second part of the applicability study is then assembled. This data consists of average speeds by section on the limited access facilities and estimates on the extent of recurrent congestion.

The traffic engineer is now in a position to estimate the total annual delay currently being experienced on the primary routes. This constitutes an estimate of the maximum possible benefit obtainable from an IMIS. A benefit/cost ratio is computed assuming all this delay can be eliminated. If the system is not cost-effective with this assumption, then the agency should not proceed any further. If the system is cost-effective with this assumption, then the next step of the procedure is performed. This step makes an estimate of the delay which can be expected be saved with IMIS operation. The analysis requires some additional effort and judgment but does not require any additional data. If the system is found to be cost-effective at this step the responsible agency is warranted in proceeding with the feasibility study. If IMIS is found not to be cost-effective at this step, the feasibility study should not be performed.

3.3 STEP 1 - QUALITATIVE GUIDELINES

A corridor to be equipped with an Integrated Motorist Information System generally should possess the following characteristics:

- At least one limited access facility (the primary route) running the length of the corridor;
- At least one other facility (limited access, service road, or signalized arterial) extending over a major portion of the corridor, and approximately paralleling the primary route;
- A geometry in which the length of the corridor is much greater than the distance between the parallel facilities. (This allows diversion without severe mileage and/or travel time penalty to the motorist);
- A corridor length of at least 5 miles (8km). Because the control center is a fixed cost regardless of mileage, per mile (km) costs normally increase as corridor mileage decreases, thereby making it more difficult to economically justify a system;
- Availability of good connector routes at least every 5 miles (8km) throughout the corridor. This requirement insures reasonable diversion capability. Connector routes spaced further apart may require diversion far in advance of a bottleneck, which tends to reduce the benefits derivable from diversion;
- Well defined termini. When the parallel routes serve common well-defined termini, diversion potential and hence system benefit is greater because of the common destinations shared by a larger number of drivers;

- Recurrent congestion. In order to consider an IMIS installation the corridor primary route(s) should be experiencing recurrent congestion. As a measure, the morning and evening peak periods should last at least 1/2 hour, with flows on the primary route at or near capacity in the peak direction during the peak period.

3.4 STEP 2 - CALCULATION OF MAXIMUM POSSIBLE BENEFIT/COST RATIO

The maximum possible benefit/cost calculation is designed to provide a quantitative estimate of the total magnitude/extent of traffic problems which can be addressed by IMIS. Total corridor delay is used as the parameter to quantify these corridor problems. The rationale for this calculation is that if the total corridor problems are of a magnitude insufficient to justify the cost of IMIS, then a feasibility study is not justified since actual system derived benefits will always be less. The calculation is given in the following sequence*:

1 Estimate annual delay caused by recurrent congestion.

The approximate delay caused by each congested section on the primary corridor routes can be computed via the following formulas:

$$Dss_i = Qi \frac{(U_{ff} - Uc)}{(U_{ff})\,(Uc)} \tag{1}$$

where

Qi = Flow, veh/lane/hour, during the peak period
U_{ff} = Free flow speed
Uc = Average speed in the congested section during the peak period
Dss_i = Delay, veh-hrs/lane mile/hour in congested section i

The total delay for each congested section is then

$$D_{T_i} = Dss_i \cdot LM_i \cdot Tc_i \tag{2}$$

where LM_i is the number of lane miles in the congested section, i, and Tc_i is the length of time per day that the section is congested. The total annual delay caused by recurrent congestion is then estimated by summing the delays of each section and multiplying by the number of days of recurrent congestion per year.

2 Estimate annual delay caused by incidents. The annual delay caused by lane-blocking incidents can be estimated as being equivalent to the delay caused by recurrent congestion. (Based on data obtained from the Gulf Freeway** and the Long Island IMIS Feasibility study.)

3 The estimated total annual delay is 1 + 2 .

- - - - - - -

*The circled numbers are keyed to specific blocks in Figure 2, the applicability flow chart

**Goolsby, M.E., "Influence of Incidents on Freeway Quality at Service," Highway Research Record Number 349, 1971

(4) The monetary equivalent of total annual delay is obtained by applying the factor used by the State for the value of a vehicle-hour of delay.* If the State does not have a specific value which it applies to similar benefit/cost studies, then $4/ vehicle hour of delay (including the fuel saving) may be used as a realistic value.

(5) Estimate IMIS Cost

Capital costs for similar systems previously developed range from about $0.5 to 1.0 million per mile ($.31 to .62 million per km) of overall corridor length. The higher figure is representative of complex corridors containing several alternate routes while the lower figure represents systems where instrumentation is confined mainly to the primary limited access route. Shorter corridors (under 10 miles (16 km) in length) tend to have a higher cost per mile, since the control center costs are distributed over fewer miles. Therefore it is recommended that $0.5 million/mile ($.3 million/km) be used as a rule of thumb capital cost for the longer, less complex corridors, while $1.0 million/mile ($.6 million/km) be used for the shorter corridors as well as for the more complex longer corridors.

The equivalent annual capital cost is computed by assuming a useful system life and applying the prevailing rate of interest with which the State is able to finance capital projects. Annual maintenance and operation costs may be assumed to average about 5 percent of the total capital cost.

Total annual costs are then computed by adding the maintenance and operation costs to the equivalent annual capital cost.

(6) Compute theortical benefit/cost ratio. This is obtained by dividing the annual monetary equivalent of delay by the annual system cost.

A sample calculation of the six steps is given below. Figure 3 illustrates a six mile (9.7km) section of 3 lane roadway (each direction). The inbound and outbound traffic flows are shown respectively for the AM and PM peak periods. It is important to note the miles of roadways which are congested and the duration of each peak period (2.5 hours assumed for this example).

*Using equations (1) and (2), the calculation for the annual delay caused by recurrent congestion, (1), is for the AM peak period***

$$Dss1 = 2000\ \frac{(55-30)}{(55)(30)} = 30.3\ veh\ hr/hr/lane\ mile$$

$$DT_1 = 30.3\ \frac{veh\ hr}{hr.\ lane\ mile}\ (3\ lanes)(1.6\ miles)\frac{(2.5\ hours)}{day} = 364\ \frac{veh\ hr}{day}$$

---

* Add the cost of 1 gallon (3.8 liters) of gasoline to the value of a veh-hr. of delay, since studies have shown this approximate fuel saving for each veh-hr. of delay saved.

**The following conversion factors apply: 1 mile = 1.61 km, units/lane mile = 0.62 units/lane km.

$$Dss2 = 2000 \frac{(55-15)}{(55)(15)} = 97 \text{ veh hr/hr/lane mile}$$

$$DT_2 = \frac{97 \text{ veh hr}}{\text{hr lane mile}} (3 \text{ lanes}) (1.8 \text{ miles}) \frac{(2.5 \text{ hours})}{\text{day}} = 1310 \frac{\text{veh hr}}{\text{day}}$$

and the PM peak period

$$Dss3 = 2050 \frac{(55-25)}{(55)(25)} = 44.7 \text{ veh-hr/hr/lane mile}$$

$$DT_3 = 44.7 (3) (1.4) (2.5) = 469 \frac{\text{veh hr}}{\text{day}}$$

$$Dss_4 = 2050 \frac{(55-22)}{(55)(22)} = 56 \text{ veh hr/hr/lane mile}$$

$$DT_4 = 56 (3) (2.0) (2.5) = 840 \frac{\text{veh hr}}{\text{day}}$$

For both peak periods combined:

$$(364 + 1310 + 469 + 840) \times \frac{5 \text{ days}}{\text{week}} \times \frac{52 \text{ weeks}}{\text{year}}$$

$$= 2983 (5) (52) = 775{,}580 \text{ veh hours/year}$$

The additional annual delay due to accidents and incidents, 2 , is equal to the recurrent delay, or 775,580 veh-hours/year

The total annual delay, 3 , is:

$$1 + 2 = 2 (775{,}580) = 1{,}551{,}160 \text{ veh hours/year}$$

The monetary equivalent benefit, 4 , is:

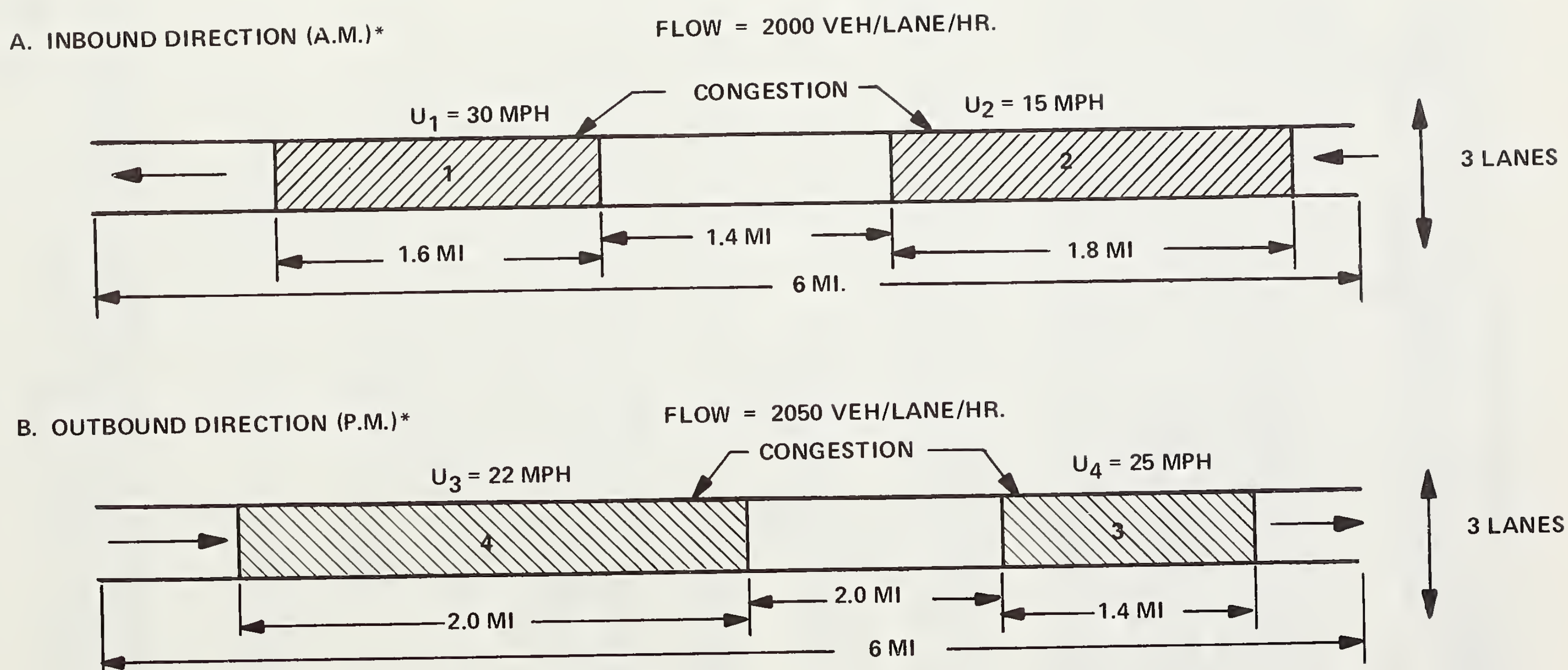
$$1{,}551{,}160 \frac{\text{veh hours}}{\text{year}} \times \frac{\$4}{\text{veh hr}} = \$6{,}204{,}640$$

The IMIS capital cost, 5 , is:

$$6 \text{ miles} \times \frac{\$0.5 \times 10^6}{\text{mile}} = \$ 3 \times 10^6$$

The assumption of a useful life of 15 years at an interest rate of 10% yields:

$$\text{an equivalent annual capital cost} = \$ 3 \times 10^6 \times 0.13147$$
$$= \$ 394{,}410$$



*OPPOSING FLOW DIRECTION ASSUMED CONGESTION-FREE.

NOTE: 1 MILE = 1.61 KILOMETERS
1 MILE/HR = 1.61 KM/HR

Figure 3. Roadway Used for Sample Calculation

and annual maintenance and operating cost $= .05 \times \$3 \times 10^6 = \$150{,}000$

so that the total annual IMIS cost $= \$394{,}410 + \$150{,}000 = \$544{,}410$

The maximum possible benefit/cost ratio, 6 *, is:*

$$\frac{\$6{,}204{,}640}{\$540{,}410} = 11.4$$

This result indicates that proceeding with the next step is warranted.

3.5 STEP 3 - CALCULATION OF EXPECTED BENEFIT/COST RATIO

The purpose of this calculation is to estimate the reduction in the total annual delay which can actually be achieved after the installation of IMIS. The reduction is computed by estimating the potential level of traffic flow which can be diverted between roadways and the corresponding level of roadway delay which is saved. The relationship of flow diverted to delay saved can be approximated as a linear relationship of the following form*:

$$\frac{\Delta Ds_i}{Dss_i} = \Delta_{D/Q} \cdot \frac{\Delta Qi}{Qi} \qquad (3)$$

where

ΔQi - is the estimated per lane volume divertible to an alternate from section i

ΔDs_i - is the actual delay saved

$\Delta_{D/Q}$ - is the sensitivity coefficient of delay saved to volume diverted. The value of this dimensionless parameter in the linear model is 4.5.

We assume that the diverted vehicles will not experience a travel time longer than had they remained on the congested roadway. Total delay saved for each congested section is obtained as follows:

$$D_{TS_i} = Ds_i \bullet LM_i \bullet Tc_i \qquad (4)$$

As previously, total delay saved, D_{TS}, is obtained by summing the delay saved from each of the section:

$$D_{TS} = \sum_i D_{TS_i} \qquad (5)$$

- - - - - - -

*This relationship is a first order linear model based on the congestion delay benefit relationship developed through simulation during the first phase of the Long Island Integrated Motorist Information System (IMIS) Feasibility and Design Study. The linear model is valid for a range of diverted volumes (ΔQ_i) in the range from 0 to 350 veh/lane/hr.

The parameter ΔQ_i (divertible volume per lane) is a difficult one to estimate because of interactive capability of the IMIS control functions. It will certainly be higher than that indicated by a static excess capacity estimation of the alternate route, since (1) ramp metering can "create" excess capacity on freeways, and (2) computerized signal control can "create" capacity on arterials. Furthermore, the corridor control algorithm in IMIS can institute upstream controls in conjunction with, or in anticipation of downstream congestion.

Therefore, diversion capability is strongly a function of the physical ability to institute the interactive controls, which in turn, depends on the available facilities. For this reason, the following guidelines, based on the Long Island IMIS feasibility study, are provided for estimating the diversion potential (without regard to present "excess" capacities):

(1) ΔQ_i = 150 veh/hr/lane, if the capability exists for

A. Ramp metering only, or
B. Diversion only, with one alternate route*

(2) ΔQ_i = 250 veh/hr/lane, if the capability exists for

A. Ramp metering with a service road only, or
B. Diversion only, with two possible alternate routes

(3) ΔQ_i = 350 veh/hr/lane, if any higher level of control is possible (e.g. ramp metering plus diversion to an alternate route).

The computation of total delay saved (D_{TS}) is computed in block ① of Step 3 of the Applicability Diagram (Figure 2). Block ② of Step 3 is equivalent to blocks ②, ③, ④, of Step 2 except that delay saved is substituted for total system delay. Finally Block ③ of Step 3 computes the equivalent benefit/cost ratio using the estimated system cost computed in block ⑤ of Step 2.

The sample calculations for blocks ①, ②, and ③, of Step 3 are given below:

① *Estimate Delay Saved –*
A. AM Peak Period

Assumed ΔQ = 250 vehicles/lane/hour, i.e., a total 750 vehicles/hour can be diverted without saturating the alternate routes.

Thus, using equation (3)

$$\Delta Ds_1 = (4.5)\left(\frac{250}{2000}\right)(30.3) = 17.0$$

- - - - - - -

*An alternate route may be a service road, a nearby arterial, or another limited a access facility.

$$Ds_2 = (4.5)\left(\frac{250}{2000}\right)(97.0) = 54.6$$

B. PM Peak Period

Assumed ΔQ = 150 vehicles/lane/hour

$$\Delta Ds_3 = (4.5)\left(\frac{250}{2050}\right)(44.7) = 24.5$$

$$\Delta Ds_4 = (4.5)\left(\frac{250}{2050}\right)(56) = 30.7$$

(2) *Estimate Annual Benefit (equations 4 and 5)*

$$\text{Annual Delay Saved} = [(17.0)(1.6) + (54.6)(1.8)]\,(3)(2.5)(5)(52)$$
$$+\ [(24.5)(1.4) + (30.7)(2.0)]\,(3)(2.5)(5)(52)$$
$$= 431{,}201 \text{ Veh-Hr/Year}$$

Additional Delay Saved (accidents/incidents) = 431,031

Total Delay Saved = 2(431,031) = 862,602

Annual Benefit = (\$4/veh-hr) (862,602)
\$3,450,408

(3) *Estimate Benefit/Cost Ratio based on Corridor Diversion Potential*

$$\frac{\$3{,}450{,}408}{\$\ \ 544{,}410} = 6.3$$

If this example had represented an actual corridor situation, it would be concluded that the corridor had "passed" the applicability guidelines, and the feasibility study should be performed.

CHAPTER 4

PLANNING AND SCHEDULING THE FEASIBILITY STUDY

4.1 INTRODUCTION

4.1.1 Objectives

- To tailor the feasibility methodology to the environment which exists in the study corridor
- To develop the study schedule
- To assess the resources (time, manpower, money) needed for the study
- To develop the plan necessary to perform the study at the resources level necessary for the specific corridor

4.1.2 Inputs

- Brief review of each task in the handbook

4.1.3 Outputs

- A work plan and schedule for completing the study
- Allocation of resources for the conduct of the work

4.2 DISCUSSION

A feasibility and preliminary design study for an IMIS corridor system will require a significant work effort. In the applicability study, an indication of the corridor size and extent was provided. There, an acceptable corridor was defined as being at least 5 miles (8km) in length with one or more limited-access roadways, as well as frontage roads and/or arterials. Since the feasibility methodology requires a substantial data base for each roadway in the corridor, a major data collection effort could be required if sufficient data are not available. Completing the study tasks can also require a substantial quantity of professional manpower. The handbook therefore includes alternative approaches and typical values, wherever possible, to provide the user with an option for limiting his manpower requirements. The actual level of effort to be expended with respect to a specific corridor will entail consideration of the quality and quantity of available data and of analytical detail versus the application of judgment and local knowledge.

To provide a framework for making these judgements, a work plan and

schedule should be formally prepared. The plan should include the specific tasks to be performed and the degree of detail that is to be followed. These items are integrated to develop a course of action for directing the overall study and for maintaining control as the project progresses. A sample manpower estimating worksheet is given in Table 1. A sample schedule form is shown in Figure 4. Typical values have been inserted in these tables to serve as general guidelines. It is anticipated that a minimal effort will require 8 to 9 man-months of labor. Planning on any significantly smaller effort runs the risk of affecting the credibility of the results. A comprehensive study, with more data collection and analysis, should require on the order of 1 1/2 man-years of effect. Substantially larger levels of effort would indicate that excessive detail is being considered (perhaps items which relate more to final design that a feasibility study). The scheduling information shown can be somewhat compressed or expanded, depending on manpower availability. It should be recognized, however, that the methodology is basically sequential, i.e., one task is followed by another, not done in parallel or with any substantial overlap. Thus, schedule compressions or expansions should be considered on a task-by-task basis.

Table 1. Manpower Estimation Worksheet
(Typical Value Shown)

Chapter Ref.	Description	Minimal Study			Comprehensive Study		
		Man-Mos. Prof.	Man-Mos. Tech	Time Interval-Mos.	Man-Mos. Prof	Man-Mos. Tech	Time Interval-Mos.
4	Planning & Scheduling The Project	0.3	-	0.2	0.5	-	0.2
5	Initial Screening of Routes	0.2	-	0.1	0.3	-	0.2
6	Assemble Corridor Data Base	0.2	0.8	0.5	0.5	4.0	1.5
7	Supplemental Analyses	0.5	1.5	1.0	1.0	3.0	1.0
8	Alternate Route Analysis	0.3	-	0.2	0.5	-	0.3
9	Select Roadways For Network Configuration	0.2	-	0.1	0.3	-	0.1
10	Establish Control Area Boundaries	0.2	-	0.1	0.3	0.3	0.2
11	Review System Function and Corridor Policy	0.1	-	0.1	0.3	-	0.2
12	Selection of Sub-system Equipment	0.2	0.2	0.2	0.6	0.6	0.4
13	Develop Alternative System Designs	0.3	0.3	0.3	0.9	0.9	0.6
14	Determine System Costs	0.5	0.5	0.5	1.0	1.0	1.0
15	Determine System Benefits	1.8	0.2	1.0	2.0	0.2	1.0
16	Benefit/Cost Evaluation of Alternative Systems	0.3	0.2	0.2	0.4	0.2	0.3
		5.1	3.7		8.6	9.9	
	Totals	8.8 man-mos		4.5 mos	18.5 man-mos		7.0 mos

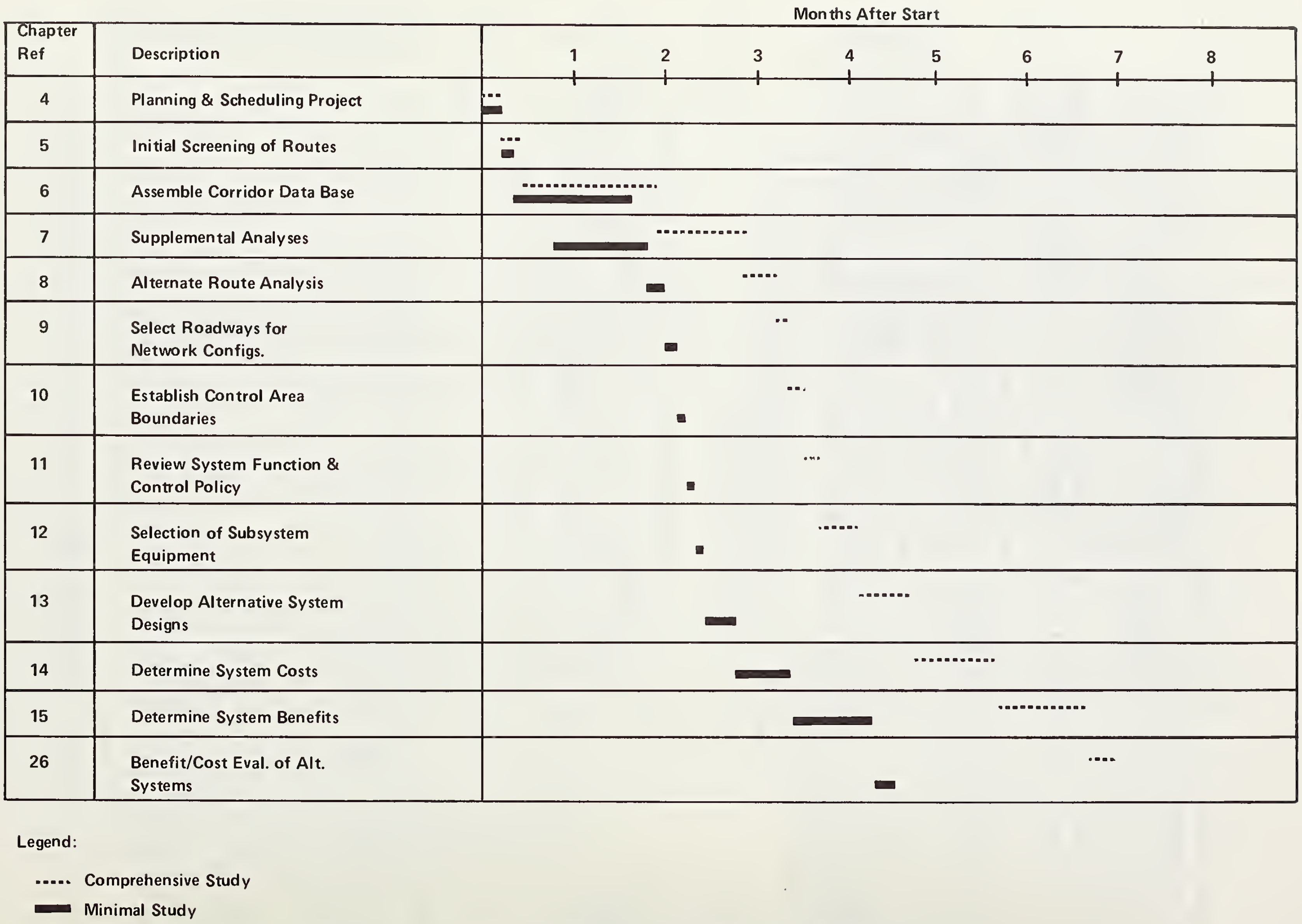


Figure 4. Feasibility Study Schedule (Typical Duration Shown)

CHAPTER 5

INITIAL SCREENING OF ROUTES

5.1 INTRODUCTION

5.1.1 Objectives

- To identify those limited access roadways, arterials and connectors that should be included in the IMIS corridor (Initial Screening).
- To prepare a baseline corridor map.

5.1.2 Inputs

- Qualitative knowledge of problem areas, capacities, route connectivity, roadway geometrics
- Appropriate maps

5.1.3 Outputs

- The candidate list of roadways for the IMIS corridor. Once the list is established, a baseline map is prepared for later use.

5.2 IDENTIFICATION OF CANDIDATE SET OF IMIS ROUTES

The purpose of this step is to identify those roadways and roadway segments which could possibly provide benefits by inclusion in the IMIS network. The inventory of candidate routes should be as inclusive as possible at this stage to avoid eliminating any prematurely.

A preliminary list should be generated from the traffic engineer's first hand knowledge of traffic operations in the corridor and the characteristics of the routes servicing the corridor. Routes which are considered to be both primary and secondary movers of corridor traffic should be included.

This list and a corresponding map could be circulated to a selected group of individuals to obtain inputs from all agencies having jurisdiction within the corridor. These individuals should be selected for their capability to address the jurisdictional aspects and identify potential problems associated with coordinating operations of roadways maintained by different agencies.

As a result of the overall recommendations made by these individuals and the traffic engineer's judgement, a list of candidate IMIS routes and preliminary corridor boundaries are generated. A typical format is shown in Table 2.

Table 2. Typical Listing of Candidate IMIS Routes and Boundaries

<u>Facilities in Direction of Corridor</u>

* Crosstown Freeway (I95)
* Inner Beltway (I495)
Crosstown Freeway Service Roads
Empire Blvd.
Kingston Ave.
Crown Road

<u>Connector Routes</u>

* Radial Freeway (I395)
Carroll Road
Linden Blvd. (State Route 9)
Smith Drive
Montague Avenue

<u>Corridor Boundaries</u>

* Inner Beltway (I495)
Roosevelt Drive
Montclair Avenue (County 94)
Crown Road
Montague Avenue

*Limited Access

5.3 INITIAL SCREENING

In this step, roadways or roadway segments are eliminated which are impractical to include or will not contribute significant utility and hence potential benefits to the Integrated Motorist Information System. The purpose of this screening is to avoid extensive data collection effort on routes which after brief review can be eliminated as candidates. Thus, available resources for data collection can be concentrated on those routes more likely to be included in the final network.

A set of screening criteria is provided to assist in the screening process. These criteria along with the experience of the traffic engineer in his specific corridor will result in the definition of a preliminary set of candidate networks for the alternative system designs. These criteria are:

- Proximity to main corridor route(s)
- Usefulness for access to ultimate destination
- Usefulness for network connectivity
- Driving quality of route
- Impact on adjoining land use
- Availability of better routes in vicinity
- Jurisdictional problems

The screening process is facilitated using a chart illustrated in Tables 3 and 4. The chart contains those characteristics listed previously which determine a route's utility for diversion. The purpose of the chart is to identify those aspects of the route which should be considered by the traffic engineer in selecting a preliminary network. Since the utility ratings in each category are still partially subjective at this stage, a route should be eliminated only when, in the traffic engineer's judgement, there is virtually no utility to be gained from its inclusion or the potential operational problems preclude its use as an IMIS route.

The descriptors to be inserted for each characteristic are excellent, good, fair, and poor with the following exceptions of "impact on adjoining land use" where "some" and "none" are more appropriate and "are there other better routes available ?" Here a simple "yes - no" will suffice.

In the example shown, three routes are listed in Table 3 and 4 along with an example set of descriptors for each. In this example, Empire Blvd. has been eliminated as a primary diversion route primarily because of its poor rating for driving quality, the availability of better routes serving the same corridor section and the negative impact on adjoining land use.

The other routes have been retained because there is no major deficiency serious enough to cause their elimination.

Table 3. Preliminary Roadway Assessment Chart, Part I

Roadway	From	To	Proximity To Main Corridor Route (s)	Utility For Major Destinations	Utility For Network Connectivity	Driving Quality	Impact of Adjoining Land Use
Empire Blvd.	I695	C94	Fair	Fair	Fair	Poor	Some
Kingston Ave.	SR 9	Smith	Good	Excellent	Good	Fair	Some
Crown Rd.	SR 9	Montague	Poor	Fair	Good	Fair	None

Table 4. Preliminary Roadway Assessment Charts, Part II.

Roadway	From	To	Are There Other Better Routes Available?	Jurisdictional Problems Associated With This Route	Other Problems Associated With Route	Route Or Segment Eliminated
Empire Blvd.	I695	C94	Yes	Route maintained by several different jurisdictions,	School zone	x
Kingston Ave.	SR 9	Smith	Yes	Signals under local control	Narrow bridge	
Crown Rd.	SR 9	Montague	No	Under jurisdiction of several different Police Depts.		

Typically, routes will be eliminated during the initial screening because of excessive distance from the primary routes, known congestion problems, availability of better routes (either corridor direction routes or connecting routes), operational problems such as school zones, busy railroad grade crossings, etc., or highly negative impact on adjacent land use.

It may be desirable to eliminate only portions or segments of a route for some of the reasons just mentioned. If removal of these segments can be accomplished without impairing the network connectivity then they should be eliminated; otherwise they should be retained for the more detailed analyses.

The result of the screening is a candidate network of roadways or roadway segments which will be further evaluated (ranked) in a subsequent task to determine which are to be retained in each alternative system design.

Having determined the IMIS roadway network, a baseline map including all candidate roadways should be prepared. The major purpose of the map (many copies of which will be made) is to provide a convenient method for compiling or summarizing data required for subsequent tasks. The map should be approximately to scale, and of sufficient size to allow data entries to be made without undue clutter. Typical entries (not necessarily on the same copy) will include distance between points, location of signalized intersections, number of lanes, travel time between points, certain IMIS field equipment locations, and the like.

CHAPTER 6

ASSEMBLY OF CORRIDOR DATA BASE

6.1 INTRODUCTION

6.1.1 Objectives

- To assemble the data elements to be used by the methodology
- To determine the adequacy of the existing data base
- To define and schedule collection of data elements considered necessary but not presently available
- To assemble the data into a form suitable for analysis

6.1.2 Inputs

- Selected data files of the organization performing the study or for whom the study is being performed
- Selected data form related organizations and authorities such as metropolitan planning organizations, police departments, and county/city/state traffic departments.
- Data collected specifically for this study

6.1.3 Outputs

- A data base in format suitable for the analysis required by the study

6.2 OVERVIEW

The methodology used in conducting a feasibility study for an IMIS project requires a substantial but selective amount of traffic data and other related information. Most procedures in the methodology require one or more data inputs. Each input may require basic data or data derived by analysis.

The Data and Methodology Flow Chart, Figure 5, graphically illustrates the relationship of the basic data elements to the uses of the data in the subsequent tasks of the methodology. Tables 5 and 6, keyed to the flow chart, provide a second summary format for the data requirements and usage. By clarifying these relationships, an understanding of why data is being collected and how it will be used is provided. This is an important point since its appreciation will reduce wasted

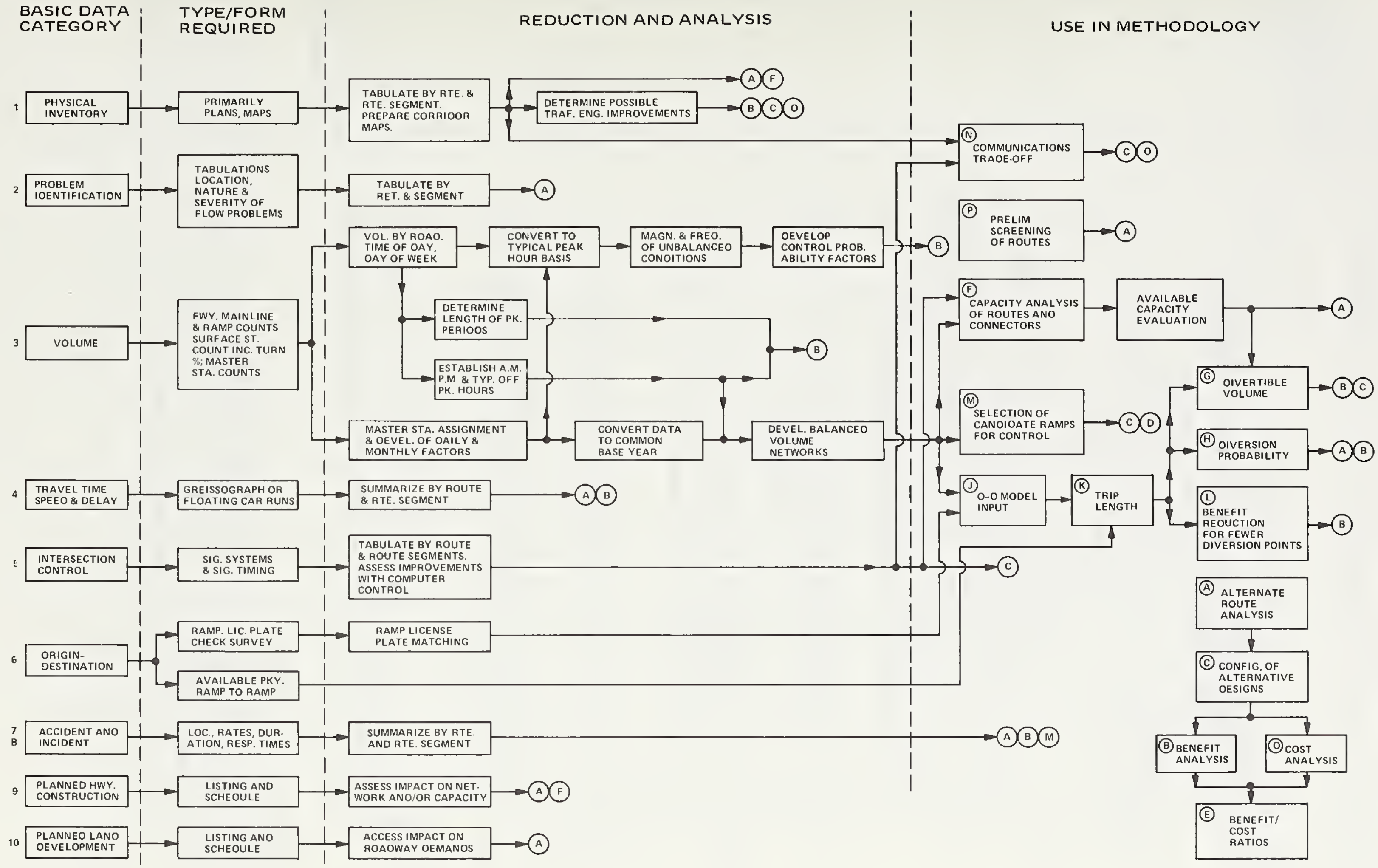


Figure 5. Data and Methodology Flow Chart

Table 5. Data Categories

	Use in Methodology (Reference Figure 5)
1. Physical Inventory Basic Data: Maps, Plans Analysis: Prepare maps, tabulations of routes, segments and roadway characteristics.	A, B, C, D, F, N
2. Problem Identification Basic Data: Location, nature, and severity of existing traffic flow problems. Analysis: Tabulations by routes and segments.	A
3. Volumes Basic Data: Freeway and ramp counts, arterial routes and intersection counts, master station counts. Vehicle classification occupancy, and pedestrian counts, if needed. Analysis: Conversion to typical peak hours, magnitude and frequency of flow variations, develop control probability factors, determine direction of peak periods, establish typical peak and off-peak volumes, convert data to common base, develop statistically balanced peak and off-peak flows for candidate networks.	B, F, J, M
4. Travel-Times, Speeds, Delays Basic Data: Floating car trips recording travel times, speeds, and delays for each freeway and arterial route. Analysis: Tabulations by routes and segments.	A, B
5. Intersection Control and Operations Basic Data: Signal system and controller inventory and timing. Other traffic control. Analysis: Tabulations by routes and intersections.	C, F, N
6. Origin-Destination Basic Data: Available ramp to ramp trips, or license plate survey. Analysis: License plate matching, ramp to ramp distribution.	J, K
7. Accidents Basic Data: Accident report summaries for routes and segments for one or more years. Special 2-week individual accident reports. Analysis: Tabulations showing accidents by number, location, type, duration, response time.	A, B, M
8. Incidents Basic Data: Special 2-week individual incident reports. Annual incident records. Analysis: Tabulations showing number of incidents by location, type, duration, response time.	A, B, M
9. Planned Highway Construction Basic Data: Listings of Programs and Schedules. Analysis: Assessment on network flow and capacity.	A, F
10. Planned Land Development Basic Data: Listings and Schedules. Analysis: Assess impact on traffic service demands.	A

Note: Inputs to the Methodology at the Use No. listed may carry through to other uses as indicated by the Methodology Table.

Table 6. Methodology Uses Requiring Data Collection

Input Required from Data Category No.	Methodology Use**	Use In Methodology	Output goes to Use**
1, 3, 5, 9	F	Capacity Analysis and Available Capacity Evaluation	A, G
*	G	Divertible Volumes	B, C
*	H	Diversion Probability	A, B
3, 6	J	O-D Model Input	K
6	K	Trip Lengths	G, H, L
*	L	Benefit Reduction for Fewer Diversion Points	B
3	M	Selections of Candidate Ramps for Control	C, D
5	N	Communication Trade-offs	C, D
1, 2, 4, 7 8, 9, 10	A	Alternate Route Analysis	C
1, 3, 4, 7, 8	B	Benefit Analysis	E
1, 5	C	Configuration of Alternate Designs	B, D
1	D	Cost Analysis	E
-	E	Benefit/Cost Ratios	-

*Inputs derived from other steps in Methodology
**Reference Figure 5

effort and cost associated with assembling the complete data base. As a general observation, all of the data requirements fall into the category of standard traffic engineering types. To the extent that field collection may be indicated for a particular corridor site, standard equipment and procedures can be used.

In the remaining sections of this chapter, each data category is treated individually, providing further description of the requirements, collection methods, and analysis. As indicated in Figure 5, these categories are:

- Physical Inventory
- Problem Identification
- Volume
- Travel Time, Speed, and Delay
- Intersection Control
- Origin-Destination
- Accident and Incident
- Planned Highway Construction
- Planned Land Development

A cost estimate for the data category is also included where possible. In each case, the write-up refers to the desirable extent of collection and analysis. It is recognized, however, that this could impose a heavy burden on the using agencies resources if the major portion of the data is not already available. Therefore, alternatives are provided, where applicable, to allow for a reduced effort, while still maintaining an adequate level of accuracy for the study.

6.3 PHYSICAL INVENTORY DATA

The operation of a traffic highway network is highly dependent upon the physical characteristics of the various segments of roadway involved. Therefore, an inventory showing locations, measurements, and other features is basic to a feasibility study of any IMIS project.

6.3.1 Uses of the Data

This data category provides the input for methodology uses A, B, C, D F, N. Thus the data become involved in:

- Alternate Route Analysis
- Benefit Analysis
- Configuration of Alternate Designs
- Cost Analysis

- Capacity Analysis and Evaluation

- Communications Trade-Off

6.3.2 Data Requirements

The physical inventory is required for:

- Preparation of maps (if not already available)

- Definition of routes and links

- Tabulations of route characteristics

- Presentation of data relative to capacity determinations

- Other graphic and physical data presentations

For capacity appraisals, it is essential to have available for each route physical data such as:

  - Number, width, and designated use of lanes

  - Horizontal and vertical curvature

  - Ramp design

  - Lane drops and merges

Additional basic physical or geometric data that will be required include:

- Roadway distances (link lengths) for travel time, speed/delay studies, and alternate route analysis.

- Geometric configuration of intersections for capacity analysis.

6.3.3 Data Collection and Analysis

Obtain as-built construction drawings from the responsible highway agency. Also, acquire available aerial photographs. Make spot field checks where more detailed information is required.

Prepare maps (as required), drawings, and tabulations for each route showing pertinent information for input and several uses in the Methodology. Maps and drawings need not be to scale; simplified diagrams may be used with appropriate notations. Table 7 and 8 show typical formats which may be used for tabulating data.

Table 7. Example of Physical Inventory Data (Section Lengths)

Section	Roadway Lengths By Sections (In Miles)						
	Route 495	GCP NSP	Route 25	Route 25A	Old Country Road	Route 25B	Route 25C
Veterans Memorial Hwy-Sagtikos Pkwy	5.03	2.09	----	----	----	----	----
Sagtikos Pkwy-Route 231	3.30	3.03	4.44	----	----	----	----
Route 231-Route 110	4.20	4.67	2.76	----	----	----	----
Route 110-Route 135	3.93	4.15	4.51	----	----	----	----
Route 135-Route 106	2.61	2.65	3.07	----	----	----	----
Route 135-Route 107	2.61	2.65	3.07	----	2.1	----	----
Route 106-Route 107	0	0	0	3.24	.5	----	----
Route 107-Glen Cove Road	4.95	5.10	4.93	2.35	----	----	----
Route 106-Glen Cove Road	4.95	5.10	4.93	----	4.83	----	----
Glen Cove Rd.-New Hyde Park Rd.	3.94	5.53	3.91	----	----	3.50	
Glen Cove Road-Herricks Road	----	----	----	----	1.87	----	----
Cross Island Pkwy-Glen Cove Rd.	----	----	----	7.94	----	----	----
New Hyde Pk Rd.-Cross Island Pkwy	3.24	3.02	2.45	----	----	2.35	2.34
Cross Island Pkwy-Clearview Expwy	1.61	1.79	----	1.13	----	1.93	2.19
Clearview Expwy-Van Wyck Expwy*	4.19	3.91	----	3.56	----	3.82	3.55

*Grand Central Pkwy for Route 495

Source: Long Island IMIS Feasibility Study

Note: 1 mile = 1.6 kilometers

Table 8. Example of Physical Inventory Data (Arterial Link Geometry)

Route	Link	Direct	Length Mi.	No. Lanes ML	No. Lanes XS	Grade % ML	Grade % XS	Trucks % ML	Trucks % XS	Remarks
L.I.E. South Service Road	Willis Ave. to Roslyn Rd.	EB	.49	3	5 2	1	0	15	10	XS-Willis Ave.
L.I.E. Sourth Service Road	Roslyn Rd. to Locust La.	EB	.24	3	5 4	0	0	15	10	XS-Roslyn Rd.
L.I.E. South Service Road	Locust La. to Glen Cove Rd.	EB	.61	3 2	3 2	0	0	15		XS-Locust
L.I.E. South Service Road	Glen Cove Rd.	EB			5 5		0		10	
L.I.E. North Service Road	Glen Cove Rd. to Locust La.	WB	.61	4 2	5 5	0	0	15	10	XS-Glen Cove Rd.
L.I.E. North Service Road	Locust La. to Roslyn Rd.	WB	.24	3 2	2 2	1	0	15		XS-Locust La.
L.I.E. North Service Road	Roslyn Rd. to Willis Ave.	WB	.49	3 2	3 5	2	0	15	10	XS-Roslyn Rd.
L.I.E. North Service Road	Willis Ave.	WB			3 5		0		10	

ML - Main Line XS - Intersecting Street

Note: 1 mile = 1.6 kilometers Source: Long Island IMIS Feasibility Study

6.3.4 Data Collection Costs

The major portion of this expense will be for putting the available data into appropriate form and for performing field checks where additional data are required. It is estimated that the cost will range from $2000 to $4000 depending upon the size and complexity of the study corridor.

6.3.5 Alternatives

The physical inventory is a basic input for the feasibility study and there are no simple alternative to obtaining it. However, if capacity analyses have already been performed, some of the detailed items such as horizontal and vertical curvature, lane widths, and other related specifics need not be compiled.

6.4 PROBLEM IDENTIFICATION DATA

The problem identification procedure will provide an appraisal of the quality of traffic flow along each route that may be considered for inclusion in the network studies. The data will set forth an inventory of existing traffic problems.

6.4.1 Uses of the Data

This data category provides input to Methodology Use A. Thus, the data become involved in:

- Alternate Route Analysis

6.4.2 Data Requirements and Analysis

Preliminary data are required for each route to locate and describe the significant traffic problems.

Included will be:

- General traffic flow conditions in peak periods and midday
- Points of congestion or constriction
- Accident rates for freeway and arterial route segments
- Other perceived problems that would affect the usefulness and effectiveness of the route.

The analytical process involves preparation of tabulations setting forth the findings for each freeway and arterial highway considered.

6.4.3 Data Collection Methods

The primary data sources will be the highway agencies and police departments having jurisdiction. TOPICS studies may provide information. For purposes of this study, the experience and judgement of the traffic operations personnel are an important input to this data category.

The basic accident information, showing numbers of accidents and accident rates for each segment can be obtained from the agency or from data that will be collected for the detailed accident data collection task described subsequently.

Field checks may be desirable or necessary to define specific congestion problems that are not disclosed or whose significance is uncertain.

6.4.4 Data Collection Costs

The cost of this data collection and analysis will vary with the mileage and complexity of the routes involved with the availability of data. It is estimated that the cost will range from $2000 to $3000.

6.4.5 Alternatives

This effort can be minimized by limiting it to qualitative judgement only and considering only major known problems. This should allow adequate treatment of the corridor roadways and not result in erroneous inclusions or exclusions.

6.5 VOLUME DATA

This is the dominant data category in terms of basic needs, analytical procedures, utility, and effort required for an IMIS study. The data will be used for various purposes in conjunction with other data categories.

6.5.1 Uses of the Data

This data category provides direct input for Methodology Uses B, F, J, and M. Thus, the data become involved in:

- Benefit Analysis

- Capacity Analysis and Available Capacity Evaluation

- O-D Model Input

- Selection of Candidate Ramps for Control

6.5.2 Data Requirements and Analysis

There are several different types of traffic volume data required and several analytical processes involved in preparing the necessary inputs to the Methodology.

A numerically balanced traffic volume flow map or tabulation is required for each freeway network studied. This will show the volumes entering or leaving the network on each freeway and at each ramp for each period studied. These periods will be for one hour on the typical weekday morning, midday, and evening conditions.

In order to develop such numerically balanced freeway network traffic flow data, it will be necessary to have traffic volumes for all links and ramps in the network. This traffic volume data, whether existing or field collected, must be converted to a common base year, a common typical day, and common hourly distributions.

Therefore, master station data must be available or developed to provide hourly, daily, monthly, and yearly factors to apply to existing or new data so as to derive the simultaneous data needed for each numerically balanced network for each study period.

A set of hourly volumes for selected locations on each corridor's limited access roadway will also be required. These data will be used to determine the magnitude and frequency of flow variations and unbalanced flow conditions during peak periods, and ultimately to develop control probability factors. The data set should consist of at least 30-40 data points with a minimum of 10 data points collected at a single location on a given roadway.

As part of the analysis, it will also be necessary to select the specific hours to be used for the numerically balanced network procedures for the morning and evening rush periods and the typical midday hour. Data showing hourly variations on typical weekdays and weekends at several locations on the network are needed to make the selections. These data are also needed to determine the durations of the peak periods for use in the benefit analysis.

The foregoing data requirements and analysis procedures also apply to a large extent to the segments of the street networks included in the study. It will be impractical to develop numerically balanced flow maps for street systems. Instead, for appraisal of intersection operations and capacity analysis, it is essential to know the volumes at key locations along the street system for the typical weekday and weekend.

Peak hour volumes for all legs of the key signalized intersections, including left turns, will be required.

6.5.3 Data Collection

The agencies responsible for highway planning and traffic operations should have substantial amounts of traffic volume information for the roadways under their jurisdictions. The extent to which the available data will be useful will be affected by format, statistical procedures, and ready availability.

For example, the AADT methodology in the planning process may utilize and summarize raw data in a form that is not readily applicable to developing the specific operational data for freeways and ramps during common peak periods. On the other hand, there are much data from continuous counting stations, intersection counts, and other specific time and location counts, that will be indispensable. The following paragraphs describe the data collection and use in greater detail.

Master Count Stations - State and local agencies with active traffic counting programs have established master count stations where volumes are recorded at regular intervals. The data are used to develop conversion factors to apply to counts at other locations to convert them to a common base year, month, day and to determine fluctuations or hourly, daily, monthly, and other time bases.

The streets and highways associated with each master station should be reviewed. If necessary, expand the coverage to include arteries where counts must be converted.

Intersection Volumes - Collect all available key intersection counts. Review to determine applicability. Use master station ratios to derive volumes for the selected analysis periods. Make additional counts where necessary to fill gaps or resolve discrepancies in available data.

Freeway Link and Ramp Volumes - Collect available counts on these roadway elements. Appraise their usefulness in developing numerically balanced data for each study network for the common periods selected.

An important consideration is that the peak period fluctuations on the freeway mainline may not be the same as at some of the ramps or segments of service roads due to the influences of traffic congestion and local peak loads. Therefore, the following data collection procedure is recommended:

- Establish a series of special temporary master count stations to represent selected sections of the freeway with similar hourly, daily, and seasonal fluctuations. Operational experience will provide the judgment for the selections. Conduct 7-day machine counts at the master count stations. Continue over a longer period if necessary to include the days on which related ramp counts are made.

- Develop a system for sampling ramp volumes for weekday AM and PM peak and midday. Use either manual or machine counts. The duration of manual counts can be selective utilizing judgment as how much time is required to obtain data satisfactory for estimating the hourly volume.

- Make sample counts at key ramps during weekend rush periods coincident with the mainline master station counts. Expansion of short counts to the desired time period can be made in accordance with standard procedures.

Service Road Volumes - Service roads are considered part of the arterial system and traffic volumes should be available from intersection counts. However, short term sample counts made in conjunction with the ramp counting procedure, will be useful at selected midlink locations.

Vehicle Classification - Vehicle classification is not ordinarily needed to determine equivalent passenger car units since mainline counts already substantially compensate by counting axles and the refinement in the volume numbers would be too small to affect the system analysis. However, factors such as percent trucks or truck types may be required in order to assess a potential alternate's route's capability to handle this type of traffic (e.g. adequate geometrics, turning radii) under diversion conditions. Thus, collection of some classification data may be necessary. Short duration sample checks at appropriate locations should be adequate for this purpose.

Special Purpose Data - The feasibility study for an IMIS project does not ordinarily require pedestrian volume or car occupancy data. High pedestrian activities are usually at intersections adequately controlled by signals. If proposed operations would add substantially to left turns through pedestrians crossing or if changes in signal timing to increase capacity may be restricted by pedestrian needs, then counts in peak hours may be desired. It is unlikely that such considerations would have significant effect upon the feasibility of an IMIS project.

Car occupancy data is not directly relevant to an IMIS feasibility study. It is the flow of vehicles that is involved in capacity and diversion considerations. If car occupancy data is desired for any location, it can be quickly and accurately obtained by very short-term sampling procedures.

Typical Samples of Volume Data - Figures 6 through 9, and Tables 9 and 10 provide typical samples of the volume data presentation and tabulation formats.

6.5.4 Data Collection Costs

Traffic volume collection costs will vary substantially depending upon the availability and utility of existing volume data.

Collection of existing data from highway agencies involves costs for acquisition and copying of records. These costs should not exceed $1,000. If for some reason data necessary for developing trends and master station factoring are not available, other sources must be investigated. The cost for this further investigation cannot be sufficiently defined for a cost estimate to be made.

Mainline volume data on freeways and arterials, if unavailable from official sources, are normally acquired by machine vehicle counters. Experience has shown that costs for collecting the mainline volume data utilizing machine counters is approximately $100 per day for both directions at one mainline location, excluding machine costs. Analysis of the field data varies depending upon the machine procedures used, but estimates of approximately 40 to 50 percent of field costs are reasonable.

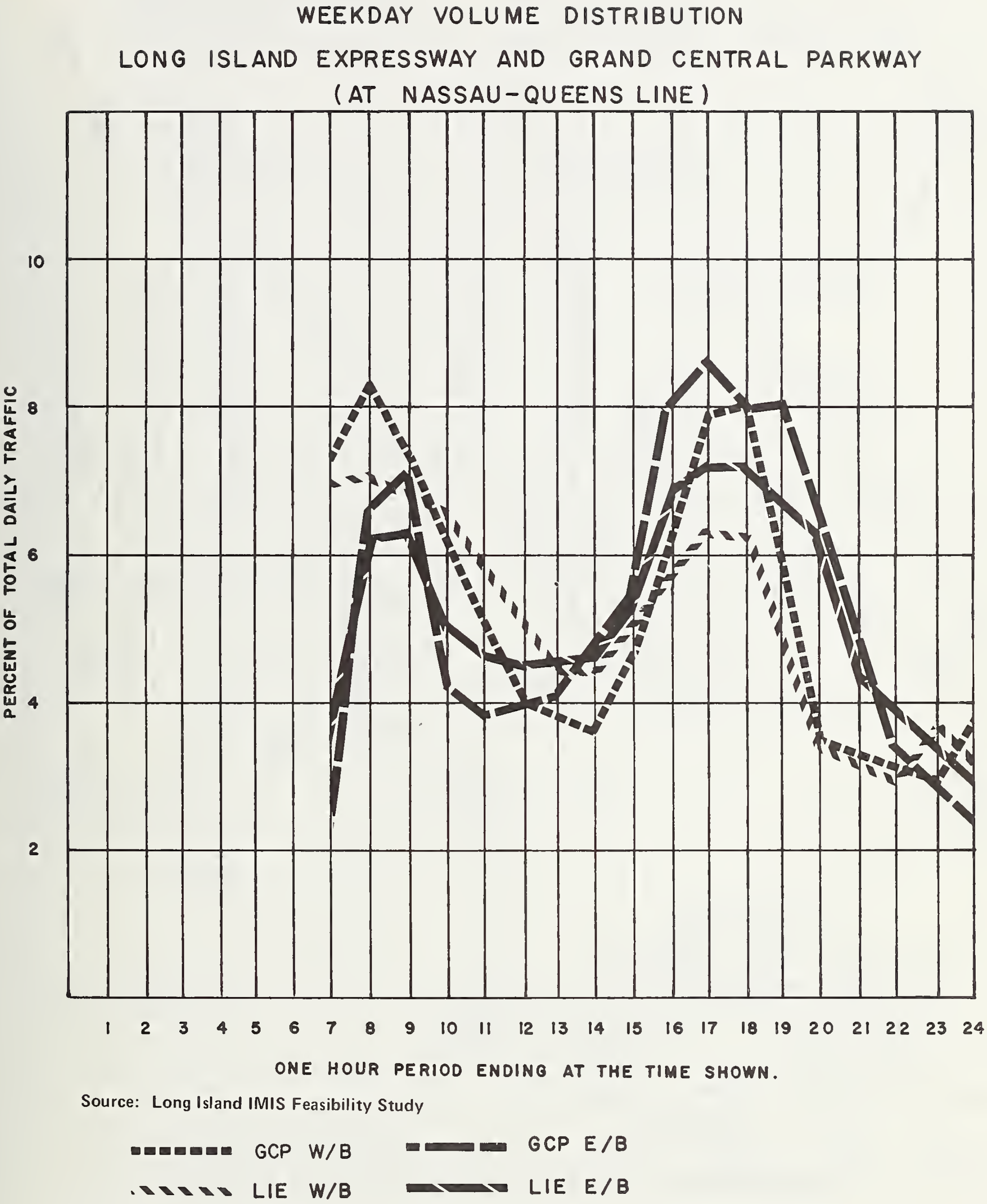


Figure 6. Example of Weekday Volume Distribution

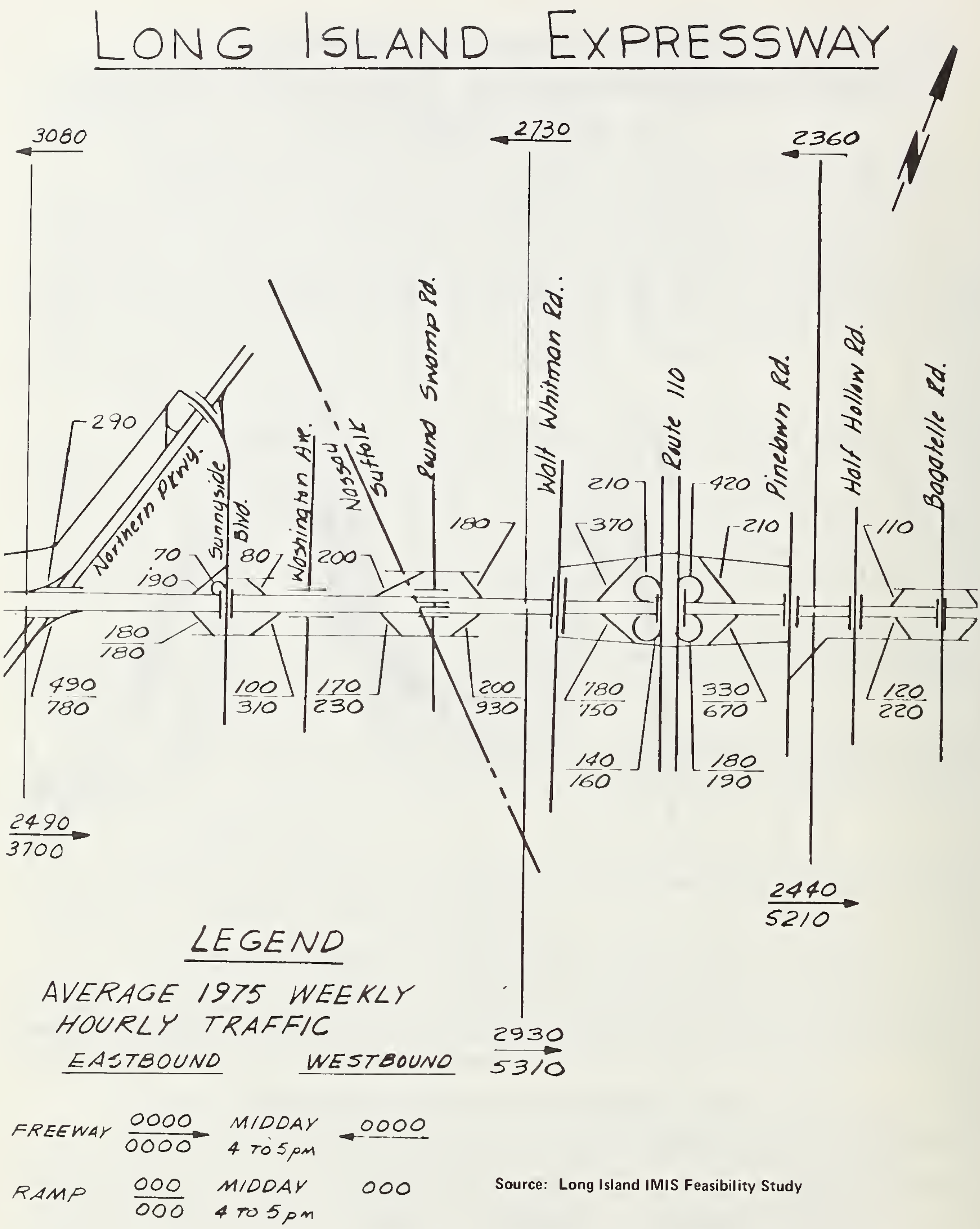
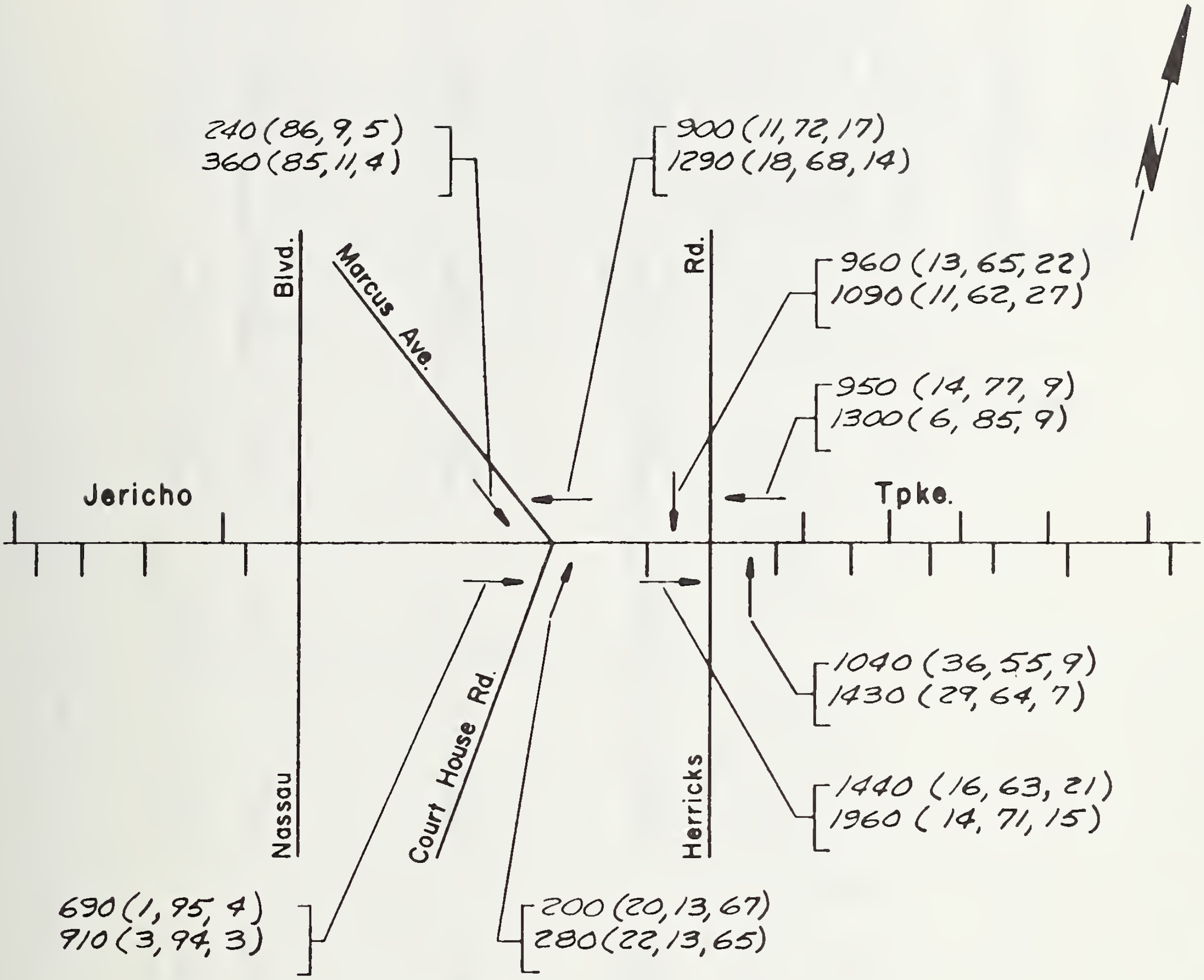


Figure 7. Example of Hourly Traffic Data (Limited Access Facility)



Source: Long Island IMIS Feasibility Study

LEGEND

Average 1975 Weekday Hourly Traffic

		Turning %
000 (0,0,0)	Midday	(Left, Thru, Right)
000 (0,0,0)	4 to 5 PM	(Left, Thru, Right)

No Scale

Figure 8. Example of Hourly Traffic Data (Surface Street)

Project No. 525

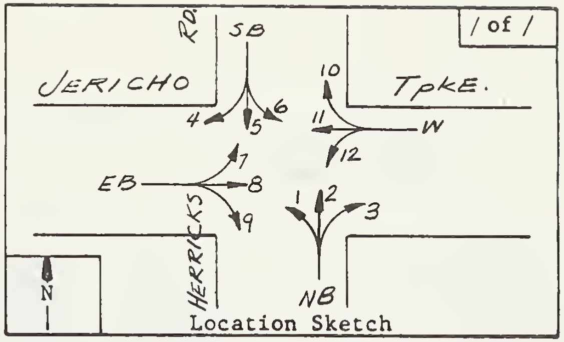
Survey INTERSECTION TRAFFIC COUNT

Location JERICHO TPKE. @ HERRICKS RD.

Observer TOM FORD Date 2/16/76 Day TUESDAY

Time 4:00 PM to 5:00 PM Weather FAIR & COLD

Notes



STREET NAME	HERRICKS RD. NORTHBOUND				SOUTHBOUND				JERICHO TPKE. EASTBOUND				WESTBOUND				
P.M. TIME	1	2	3	Total Appr.	4	5	6	Total Appr.	7	8	9	Total Appr.	10	11	12	Total Appr.	Inter-section Totals
4:00-4:15	83	204	17	304	27	169	53	249	50	291	44	385	18	259	26	303	1241
4:15-4:30	102	249	28	379	31	192	66	289	61	379	69	509	21	271	30	322	1499
4:30-4:45	98	236	31	365	30	181	79	290	78	362	77	517	24	268	29	321	1493
4:45-5:00	132	226	24	382	32	134	96	262	85	360	104	549	15	307	32	354	1547
Total Appr.																	
Total Appr.																	
Total Appr.																	
Peak Hour																	
Hour Total	415	915	100	1430	120	616	294	1090	274	1392	294	1960	78	1105	117	1300	5,180

Source: Long Island IMIS Feasibility Study.

Figure 9. Example of Intersection Volume Data

Table 9. Example of Master Station Assignment

Parallel and Connector Routes	Highway Section	Master Station
Willis Avenue	Old Country Road to Northern State Parkway Northern State Parkway to 25A	Old Country Road Station Northern Boulevard Station
Roslyn Road	Old Country Road to Northern State Parkway Northern State Parkway to 25A	Old Country Road Station Northern Boulevard Station
Glen Cove Road	Long Island Expressway to 25A	Northern Boulevard Station
Route 106	All	South Oyster Bay Road Station
Route 107	All	South Oyster Bay Road Station
Broadway	All	South Oyster Bay Road Station
South Oyster Bay Road	All	South Oyster Bay Road Station
Manetto Hill Road	All	South Oyster Bay Road Station
Round Swamp Road	All	South Oyster Bay Road Station
Route 110	Long Island Expressway to Jericho Turnpike Jericho Turnpike to 25A	Route 110 Station Pulaski Road Station
Deer Park Avenue	All	Suffolk County Veterans Memorial Highway Station

Source: Long Island IMIS Feasibility Study

Table 10. Example of Volume Adjustment

ARTERY 25A - NORTHERN BLVD

BASE YEAR 1975

LOCATION	DATE	DAY	TIME	VOLUME	YEARLY FACTOR	DAILY FACTOR	MONTHLY FACTOR	ADJUSTED VOLUME
BRIDGE E/B	4·—·77	TUES.	1-2	1175	1.00	.99	1.02	1190
" "	"	"	4-5	2014	1.00	.99	1.02	2030
BRIDGE W/B	"	"	1-2	1283	1.00	.99	1.02	1300
"	"	"	4-5	1357	1.00	.99	1.02	1370
STATION 020 E/B	6·13·74	THUR.	1-2	650	1.05	.96	.89	580
"	"	"	4-5	1410	1.05	.96	.89	1260
STATION 020 W/B	"	"	1-2	1080	1.05	.96	.89	970
"	"	"	4-5	1330	1.05	.96	.89	1190
STATION 021 E/B	6·12·74	WED.	1-2	780	1.05	1.02	.89	740
"	"	"	4-5	1220	1.05	1.02	.89	1160
STATION 021 W/B	"	"	1-2	860	1.05	1.02	.89	820
"	"	"	4-5	1070	1.05	1.02	.89	1020
STATION 022 E/B	6·13·74	THUR.	1-2	910	1.05	.96	.89	820
"	"	"	4-5	1470	1.05	.96	.89	1320
STATION 022 W/B	"	"	1-2	480	1.05	.96	.89	430
"	"	"	4-5	600	1.05	.96	.89	540
STATION 023 E/B	8·29·74	THUR.	1-2	520	1.05	.96	.91	480
"	"	"	4-5	740	1.05	.96	.91	680
STATION 023 W/B	"	"	1-2	600	1.05	.96	.91	550
"	"	"	4-5	630	1.05	.96	.91	580

Source: Long Island IMIS Feasibility Study

Manual traffic volume counting procedures are utilized for acquiring ramp, and intersection volumes and for special purpose counts.

The cost for a two-man count crew to perform a volume study is approximately $40 for one study period. The AM peak, midday, PM peak on a weekday, and weekend peak, would comprise four study periods.

Costs of the analysis of the existing and new data collected so as to numerically balance the study networks and to develop the tabulations, graphics, and presentations for use in the Methodology will vary widely depending upon the availability, completeness, format, and quality of existing data. The cost for the analytical procedures when most of the data used is field collected can be estimated at one and one-half times the field data collection costs.

6.5.5 Alternatives

One approach for significantly reducing the data collection and analysis efforts is to consider only one of the peak periods and assume it is representative of both (normally a reasonably good assumption). Also, if its traffic is generally light, the off-peak period might be neglected since this would not provide a substantial amount of system benefits.

Further simplifications would be dependent on the agencies' ability to estimate (rather than collect) certain types of data, based on their working knowledge of the corridor roadways.

6.6 TRAVEL TIME, SPEED, AND DELAY DATA

The travel time/speed delay category provides an important source of data which describes the mean performance levels of the corridor roadways during the peak and off-peak travel periods. In terms of overall importance to the study the travel time/speed delay category ranks with the development of the volume/flow data and accordingly should receive equivalent attention.

6.6.1 Uses of the Data

The travel time, speed, and delay data provide input for Methodology Uses A, and B. Thus the data become involved in:

- Alternate Route Analysis

- Benefit Analysis

6.6.2 Data Requirements and Analysis

Travel time, speed, and delay data are required during peak hours and typical midday periods for all freeways and arterials under consideration.

In the analytical process, the data will be used for each route segment or link to determine:

- Travel Time
- Trip Speed
- Operating Speed
- Stops and delays at signals
- Key congestion points
- Severity and duration of congestion

Tabulations and graphs can be used to set forth the results of the analytical procedures. Typical examples are provided in Tables 11 through 13 and Figure 10.

In the analysis of travel time, speed, and delay runs, the time measures are converted into average or mean speeds. Summary statistics can be developed for various segments between selected control points as well as for the entire study route.

There are many unpredictable variables that will affect the travel time, speed, and delay recorded on any trip. Generally, a range in travel speed of plus or minus 10 percent from average, is to be expected. This range is sufficiently accurate for the purposes of the IMIS study. Of course, the more runs, the greater the degree of reliability. See Table 7-1 in the Manual of Traffic Engineering Studies, Fourth Edition, Institute of Transportation Engineers, 1976, for criteria to obtain a confidence level of 95 percent.

6.6.3 Data Collection

Review any travel time studies that have been made by the highway agency. Make floating car runs at appropriate times covering the freeways and arterial streets under study.

Typically three trips in each direction for each time-of-day period should provide sufficient data for the analysis; however, a larger sample size could be required if variations between trips are large.

Control points should include all ramps on freeways and key intersections on arterials.

6.6.4 Data Collection Costs

The floating car run requirements for this type of study are relatively simple. Manual techniques using prepared forms listing the recording points will be satisfactory and less expensive than graphic recording techniques both in field and office procedures.

Table 11. Example of Travel Time, Speed and Delay Data

Route L.I.E. So. SERVICE ROAD   Direction ☐NB ☐SB ☑EB ☐WB

Trip Started At 4:45 AM (PM)   AT GRAND CENTRAL PKWY. (Location)   38,132.1 mi/ km (odometer reading)

Trip Ended At 5:27 AM (PM)   AT BROADWAY, RTE. 107,106 (Location)   38,150.3 mi/ km (odometer reading)

______ Peak

Note: Enter travel time & distance at connector routes in sequence under location.

Location	TIME		REMARKS (SECONDS STOPPED, ETC.)	SEC. STOP	TRAVEL TIME
1 GRAND CENTRAL PKWY. ℄	:00				
2 VAN WYCK EXPWY.	:30	:30		15	:15
3 MAIN ST.	2:40	2:10		40	1:30
4 KISSENA BLVD.	4:35	1:55		10	1:45
5 164th ST.	6:00	1:25		40	:45
6 UTOPIA BLVD.	7:55	1:55			1:55
7 FRANCIS LEWIS BLVD.	9:50	1:55			1:55
8 CLEARVIEW EXPWY.	10:25	:35			:35
9 BELL BLVD.	11:35	1:10		40	:30
10 SPRINGFIELD BLVD.	12:50	1:15			1:15
11 EAST HAMPTON BLVD.	14:05	1:15			1:15
12 CROSS ISLAND PKWY. ℄	14:20	:15			:15
13 DOUGLASTON PKWY.	15:45	1:25			1:25
14 MARATHON PKWY.	16:30	:45		25	:20
15 LITTLE NECK PKWY.	18:20	1:50	D P	25	1:25
16 LAKEVILLE RD.	20:15	1:55			1:55
17 COMMUNITY DR.	20:40	:25		15	:10
18 NEW HYDE PARK RD.	22:00	1:20		60	:20
19 SHELTER ROCK RD.	24:55	2:55		50	2:05
20 SEARINGTOWN RD.	26:50	1:55		35	1:20
21 MINEOLA BLVD./WILLIS AVE.	28:35	1:45		10	1:35
22 LOCUST LA.	30:15	:40		5	:35
23 GLEN COVE RD.	32:30	2:15	TS	1:20	.55
24 POST AVE.	35:10	2:40		5	2:35
25 JERICHO TPKE.	37:45	2:35		10	2:25
26 BROADWAY, RTE. 106 & 107	41:50	4:05		60	3:05
27					
28					
29					
30					
TOTALS 18 Stops	41:50				

TOTAL TRIP DISTANCE 18.2 mi 29.3 km
TOTAL TRIP TIME 69.7 % of hr.
AVERAGE SPEED 26.1 mi/hr (42 km/hr)

RUNNING SPEED 33.5 mph
(total trip distance) mi÷(total trip time - total stopped time) hrs.
Seconds/Stop 29
Stops/Mile 10.1
Stops/km 16.3

SYMBOLS OF DELAY CAUSE	TS=Traffic Signal	SS=Stop Sign	LT=Left Turn
	A=Accident	DV=Disabled Vehicle	RN=Rubber Necking
B=Bus Loadings	C=General Congestion	DP=Double Parking	PED=Pedestrians

Source: Long Island IMIS Feasibility Study

Driver K. ZABIELSKI   Date 3/17/76
Recorder K. ZABIELSKI   Weather CLEAR

Table 12. Example of Average Speed Data

AVERAGE SPEED BY SECTION - EASTBOUND PM PEAK (4-5PM)

	Long Island Expressway Route 495		Long Island Expressway N. Service		Northern State Parkway		Route 25		Route 25A		Old Country Rd		Route 25B		Route 25C	
	IMIS	TOPICS	IMIS	TOPICS	IMIS	TOPICS	IMIS	TOPICS	IMIS	TOPICS	IMIS	TOPICS	IMIS	TOPICS	IMIS	TOPICS
Vets Mem Hwy - Sagtikos Pkwy	22	43	-	-	52	50	-	-	-	-	-	-	-	-	-	-
Sagtikos Pkwy - Route 231	31	60	-	-	54	55	26@@	28	-	-	-	-	-	-	-	-
Route 231 - Route 110	33	38	-	-	47	53	26	31	-	-	-	-	-	-	-	-
Route 110 - Route 135	29	26	-	-	40	34	28	34	-	-	24#	-	-	-	-	-
Route 135 - Route 106/107	35	30	-	-	32**	-	18	27	-	-	20	17	-	-	-	-
Route 106 - Route 107	-	-	-	-	-	-	-	-	40	37	11	16	-	-	-	-
Route 106/107 - Glen Cove Rd	31	36	33	42¢¢	32**	35	-	30	-	31	-	19	-	-	-	-
Glen Cove Rd - Willis/Mineola	33	-	28	-	41	-	-	-	-	-	-	-	-	-	-	-
Willis/Mineola - New Hyde Park	26	-	29	-	39	-	-	-	-	-	-	-	-	-	-	-
Willis/Mineola - Lakeville Rd	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
New Hyde Park Rd - Lakeville Rd	19	-	25	-	25	-	17	-	-	-	-	-	-	-	25	-
Lakeville Rd - Little Neck Pkwy	-	-	-	-	28	-	-	-	-	-	-	-	-	-	27	-
Lakeville - Cross Island Pkwy	28	-	26	-	29*	-	16	-	20	-	-	-	19¢	25¢	25*	23*
Cross Island - Clearview Exp	24	48	22	-	46	50	-	-	20	18	-	-	29	21	31	27
Clearview Exp - Van Wyck Exp	34	36	21	-	30	36@	-	-	17	23	-	-	13	13	30	19
Van Wyck Exp - Grand Central	17	-	50	-	47***	-	-	-	-	-	-	-	-	-	-	-

* Little Neck Parkway - Cross Island Parkway
** Route 135 - Glen Cove Road
*** Van Wyck Expressway - Long Island Expressway
\# Long Island Expressway - Route 135

@ Clearview Expressway - Long Island Expressway
@@ To Route 231 East (Deer Park Road)
¢¢ Jericho Turnpike - Glen Cove Road
¢ New Hyde Park Road - Cross Island Parkway

Source: Long Island IMIS Feasibilty Study

Table 13. Example of Delay Due to Stops

NUMBER OF STOPS BY SECTION - AM PEAK (7-8AM)

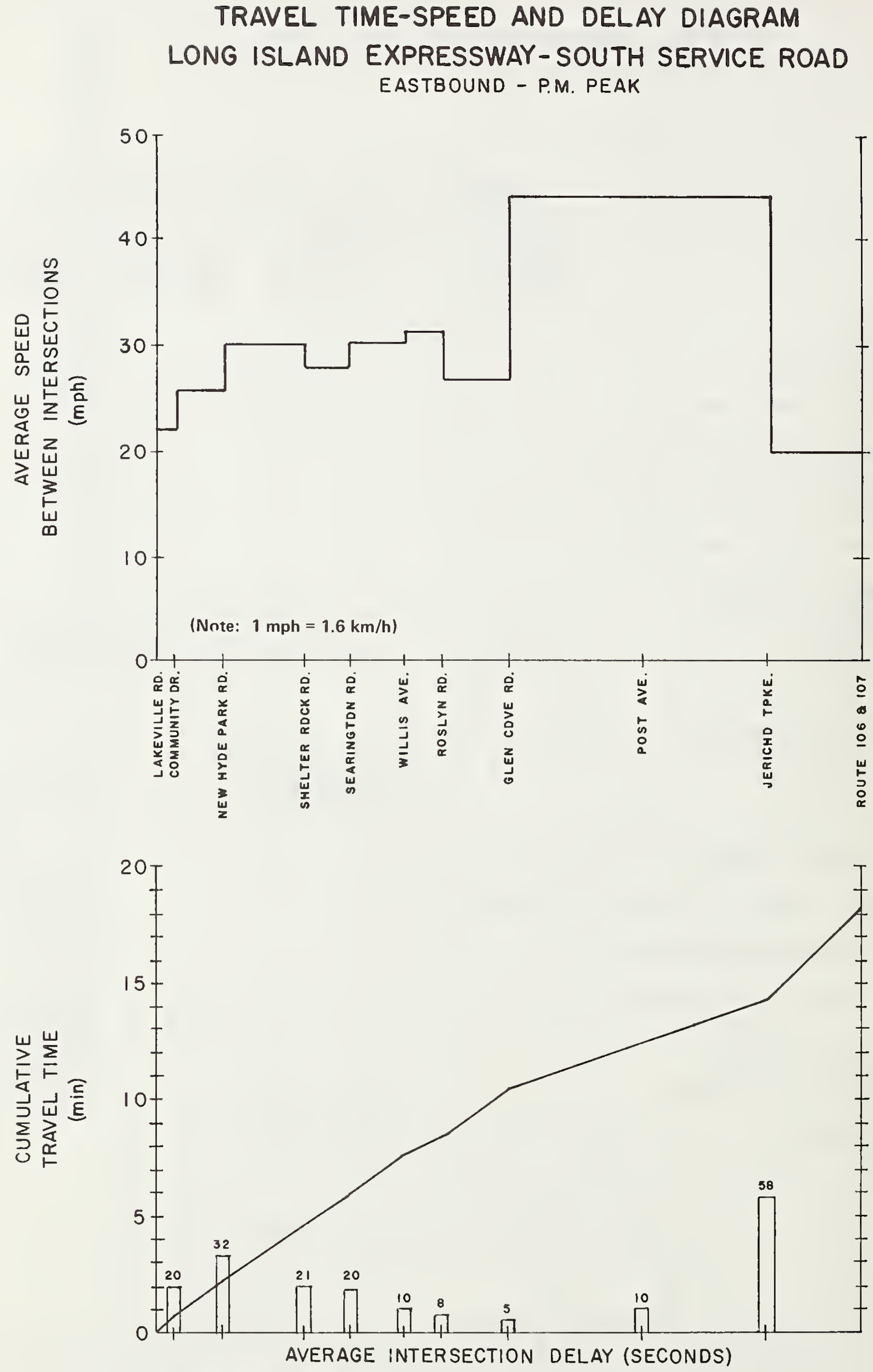
	Route 106/107		Round Swamp Road		Route 110		Route 231	
	NB	SB	NB	SB	NB	SB	NB	SB
25A - Jericho Turnpike	2	1	-	-	-	-	-	-
Jericho Tpke - Long Island Exp	1	1	-	-	-	-	-	-
Jericho Tpke - Northern State Pkwy	-	-	0	0	1	1	1	1
Long Island Exp - Northern State	0	0	1	1	2	3	2	2
Northern State - Old Country Rd	3	2	0*	0*	-	-	-	-
Total Number of Stops	6	4	1	1	3	4	3	3
Total Stop Time (Seconds)	185	135	15	12	70	75	67	80
Second/Stop	30.8	33.8	15.0	12.0	23.3	18.9	22.3	26.7
Distance (Miles)	5.8	5.8	2.9	2.9	3.5	3.5	2.9	2.9
Stops/Mile	1.0	.7	.4	.4	.9	1.1	1.0	1.0

* Long Island Expressway - Old Country Road

	Veterans Memorial Highway	
	NB	SB
Jericho Turnpike - Sagtikos State Parkway	1	1
Sagtikos State Parkway - Northern State Parkway	0	0
Northern State Parkway - Route 111	2	2
Route 111 - Long Island Expressway	1	1
Total Number of Stops	4	4
Total Stop Time (Seconds)	55	75
Second/Stop	13.8	18.8
Distance (Miles)	6.2	6.2
Stops/Mile	.7	.7

(Note: 1 stop/mile = 0.62 stops/Km)

Source: Long Island IMIS Feasibility Study



Source: Long Island IMIS Feasibility Study

Figure 10. Example of Travel Time, Speed and Delay Diagrams

On this basis, it is estimated that the cost for data collection and analysis will approximate $50 per trip.

6.6.5 Alternatives

As in the case for volume data, substantial savings may be obtained by considering only one peak hour as representative for both, and excluding the off-peak hour runs. Also, since travel time or average speed provides a general measure of route quality, these parameters should be adequate for the feasibility study (i. e. recording of stops and delays could be omitted).

6.7 INTERSECTION CONTROL AND OPERATION DATA

Traffic signals exert a great influence upon the flow of traffic at an intersection, along an arterial, and in a street network. Therefore, thorough knowledge and appraisal of existing and feasible capabilities and operations of signals is essential to estimates of how much more traffic can be carried by the arterial streets under various traffic flow conditions.

6.7.1 Uses of the Data

The Intersection Control and Operation Data Category provides the input for Methodology Uses C, F, and N. Thus, the data become involved in:

- Analysis of available capacity

- Configuration of Alternative Designs

- Communications Trade-Off

6.7.2 Data Requirements and Analysis

An inventory is required at key locations on alternate routes in order to determine the types and the capabilities of signal systems and of the intersection control equipment. Signal operation and timing, locations of stop or yield signs, and appraisal of other factors affecting capacity, are required.

Intersection control data are required for all key locations which significantly affect the capacity of the arterial route. Normally, these will include intersections of two arterial routes, multi-phased signals, the locations where left-turns will be increased, and others which may be indicated by Data Categories 1 and 2.

The capacity analysis requires input of effective green signal time per hour. Tabulations for each route are required showing amount of arterial green signal time available for through traffic per hour. This can be calculated from the signal timing data as a percentage of total time. Percentages of green time for other traffic phases will also be useful in appraising possible traffic signal control improvements.

Traffic engineering analysis of the signal systems and key intersections is required in order to appraise the need or usefulness of traffic control improvements. Because of the various traffic signal control configurations, (i.e., fixed-time, semi or full-vehicle actuated, density or gap reduction, and varied signal system configurations), the personnel assigned to acquiring or analyzing this data should be well versed in the operations and traffic flow effects of these varying types of signal controllers.

6.7.3 Data Collection Methods

Existing records kept by the agencies responsible for traffic signal control and operations are usually sufficient for inventory at most locations. If not available, field inventory will be necessary. For an example of data format see Table 14. For other typical forms see Appendix D in the Traffic Control Systems Handbook, USDOT, 1976.

Field inspections by qualified traffic engineers will be required to appraise the efficiency of existing operations and to assess the need and feasibility of signal and other traffic improvements.

6.7.4 Data Collection Costs

Collection of existing data, field inventory, traffic operations appraisals, and analytical effort is estimated to cost from $2,000 to $4,000 for a system study up to 100 intersections. The costs for larger studies should increase proportionately.

6.7.5 Alternatives

There are no simple alternatives to obtaining the required intersection control data.

6.8 ORIGIN - DESTINATION DATA

Origin-Destination as used in this study relates to point of entry and point of departure from the freeway, rather than beginning and end of trip.

6.8.1 Uses of the Data

This data category provides the input for Methodology Uses J and K. Thus, the data become involved in:

- Origin-Destination Model

- Trip Length (Ramp to Ramp)

Table 14. Example of Signalization Data

INTERSECTION L.I.Exp. No. Service Rd. (N.S.R) @ Glen Cove Rd.

TYPE OF CONTROLLER 2 PHASE S VA: — PHASE FT.

INTERSECTION APPROACH	GLEN COVE RD.	N.S.R.					
PHASE	A	B					
DETECTOR SPACING	NA	90'					
MIN. GREEN	(10) NA	13					
MAX. GREEN #1	NA	30					
MAX. GREEN #2	NA	(NOT USED)					
AMBER CLEAR	4	3.5					
RED CLEAR	1.5	1					
RECALL	NA	OFF					
MIN. PHASE TIME / SPLIT	62.5, (15.5) / 78%	17.5 / 22%					
MAX. PHASE TIME / SPLIT	45.5 / 57%	34.5 / 43%					

COORDINATION EQUIPMENT: 80 SEC. BACKGROUND CYCLE

TYPE: SYNCHRONIZER (MASTER) 58% OFF-SET @ END PHASE A GREEN
CH PC DMS-1

REMARKS: THIS INTERSECTION IS PART OF A SIMPLE SYSTEM FOR THREE (3) INTERSECTIONS WHICH PROVIDES A CONTINUOUS N.B. OFFSET

OFFSET	LOCATION
49% R1 50% R2	THE PINES SIGNAL (— FT. NORTH OF N.S.R.
58%	NO SERVICE RD.
60% 61%	SO. SERVICE RD.

SOURCE: Long Island IMIS Feasibility Study

6.8.2 Data Requirements

Ramp to ramp trip distribution data for peak periods on typical weekdays and weekends are required for the freeway segments under study. This requirement can be met by data which show for each entrance ramp the distribution to downstream exit ramps. Conversely, data for each exit ramp which show distribution from the upstream entrances can be used.

6.8.3 Data Collection Methods

Existing traffic operations or planning studies conducted by highway or planning agencies may provide the necessary data from which ramp to ramp distributions can be derived. However, general O-D studies usually are not specific enough to be of use for an IMIS feasibility study. Therefore, it is probable field data will have to be collected.

One method effective for this specific purpose is by driver interview at busy exit ramps. A simple question such as "where did you get onto the freeway" can be asked and the answer marked on a tally sheet listing by name and number all points of entry to the freeway section. A one hour sample or 200 vehicles will provide sufficiently accurate information for an IMIS feasibility study. This method requires availability of interview locations where a large proportion of the vehicles are stopped such as at STOP signs or signals. This method has the advantage of very rapid data processing. A potential disadvantage is that drivers may object to the inconvenience of being stopped. Safety could also be a potential problem.

A second method is to use a license plate survey. Here, observers are stationed at each exit and entrance ramp to obtain the license plates of the vehicles using the ramp. Typically, the observers use cassette tape recorders for this purpose. The data are subsequently transcribed and a computer is used to "match" licenses plates. (Matching can be done manually, but this becomes impractical when a large number of ramps are involved.) The major drawback of this approach is the high cost due primarily to the extensive manpower required.

A third method is to use an "O-D model" and check its reasonableness with a limited license plate survey. The only data requirements for the model are the balanced freeway network diagrams which are developed anyway during the feasibility study. The model produces the necessary ramp component data by apportioning fractions of the mainline link and ramp flows as it progress down the freeway. All computations can be performed with a hand calculator. (A detailed description of the model and its use is provided in Chapter 7.)

For spot-checking the O-D model select two or three high-volume entrance ramps near the beginning of the freeway study section and two or three high-volume exit ramps towards the end of the section. Station one or two observers at each ramp, dependent upon ramp volumes. A study period of $1\frac{1}{2}$ to 3 hours should be used in order to assure peak hour coverage at each location. Record license plate numbers using cassette tape recorders. Send a lead car through the system

in order to provide a time frame or "window" for number matching and analytical purposes. If the ramp volumes are low, record numbers on tally sheets so as to reduce matching time.

Typical instructions for data collection are shown in Table 15.

Table 15. Typical Instructions For O-D Calibration Survey

Ramp Locations	Start. Time
N/B Meadowbrook to W/B Northern State Pkwy.	7:30 A.M.
W/B Northern State Parkway to W/B L.I. Express.	7:30 A.M.
Long Island Expressway to Cross Island Parkway	7:35 A.M.
Long Island Expressway to Grand Central Parkway	7:40 A.M.

LICENSE PLATE MATCHING SURVEY

INSTRUCTIONS TO OBSERVERS

1. Survey will be conducted Monday morning March 8, 1976 weather permitting.
2. Survey will not be conducted in rain, sleet, or snow.
3. Report to your assigned location fifteen (15) minutes prior to start time.
4. Check out all equipment.
5. Make sure you have pencils, pad, clipboard, and watch, in case tape recorder malfunctions.
6. Label casettes - Location, observer, side #.
7. Start study at start time.
8. Do not leave your location or stop collecting data until five (5) minutes after final travel time test car passes your location.
9. Read in license plate numbers and vehicle classification. For passenger cars, read in license plate number only.
10. Read time into tape recorder every 2 minutes.
11. Check the amount of tape remaining.
12. Watch battery level.
13. Try to keep recorder warm.

Table 15. Typical Instructions For O-D Calibration Survey (Continued)

14. Read in time, location, observer, and side number of tape at start of each side of each cassette.

15. If you miss a vehicle's license plate say "Missed", but if possible give a description (e.g., "blue ford sedan).

16. If recorder malfunctions, keep a count of all vehicles and write down as many plate numbers and times as possible. Ask for a new recorder when your supervisor arrives (if possible).

17. Speak clearly and distinctly.

18. Report all problems to your supervisory (if possible - turn on lights or use other technique to signal a problem).

19. Do not do anything to attract attention to yourself as it will cause drivers to "rubberneck".

20. Adjust your location so you can best read the license plate numbers and are in a safe location.

21. Experience has shown it best to face approaching traffic and read the front plate thereby allowing a second chance to read the rear plate if required.

22. If truck plates cannot be read describe the truck by color, and name of company (if any) and anything unusual about it.

23. Record time test car passes your location.

24. Set watches by commercial radio or some other accurate means. Time must be within one or two minutes for all observers.

EQUIPMENT FOR O & D OBSERVER

The observer should be equipped with the following:

1. Cassette tape recorder with batteries, including one spare set of batteries.
2. Cassette tapes to cover time of study, plus one spare cassette.
3. Pencils
4. Pads

(In case of malfunction)

5. Watch

The area Supervisor shall have extra tapes and batteries in case difficulties arise.

6.8.4 Data Collection Costs

The field data collection cost is estimated to be in the $500 to $1500 range for the limited O & D license plate check described (i.e. 2 to 3 entrance ramps and 2 to 3 exit ramps).

The driver interview survey is estimated to cost between $3000 and $4000 for about 50 ramps.

6.8.5 Alternatives

The alternative of using the O-D Model has already been mentioned above. The only further reduction in effort could be the acceptance of the O-D model results without performing the check survey. The O-D model has been checked in several studies against full license plate survey results, and generally found to be accurate to an order of about 10 percent, which should be acceptable for the feasibility study.

6.9 ACCIDENT/INCIDENT DATA

Accident and incident (e.g., breakdowns, spilled loads, etc.) data for an IMIS Study are used for measuring the impact of these occurrances upon traffic flow, assessing route quality, and estimating system benefits.

6.9.1 Uses of the Data

This data category provides the input for Methodology Uses A, B, and M. Thus, the data become involved in:

- Alternate Route Analysis

- Selection of Ramps for Control

- Benefit Analysis

6.9.2 Data Requirements

For each link in the IMIS study network, it is necessary to have basic information on numbers, locations, day of week, times of day, and types of occurrence. In addition, data are needed relative to the severity, response times or emergency services, and durations of the traffic interferences.

6.9.3 Data Collection Methods

A. Accident Data

Collect the basic information on a location, time, etc. from the agencies with uniform coverage and comprehensive reporting system. The Departments of

Motor Vehicles, Transportation, and Police are the normal sources. Data covering at least the latest available 12-month period are required. Data for two-year or three-year periods will refine the analysis.

For freeways, data on the severity of the impact upon traffic flow durations, and response are not usually available in consistent forms. Possible sources are the police and towing services. Review of their reports on each case can be difficult and unreliable in various degrees. Therefore, it is recommended that such information be gathered as part of a special 2-week study (combining accident and incidents).

This 2-week study (for freeways) involves a report on each event that affects traffic flow or involves motorist services. The reports are made by the police responding to the incident. See Figure 11 for a typical report from which may be used.

B. Incident Data

Inasmuch as incidents are not legally required to be reported, there are no established sources of information. Police and emergency service vehicle logs may be useful. The above noted 2-week study is perhaps the best source of data for the freeway system. (Incidents on the arterials are not included in the IMIS study.)

6.9.4 Data Analysis

Identification of high accident locations should be cataloged with respect to each roadway in the corridor. For arterials, these locations are generally in the vicinity of intersections. Data can thus be summarized by intersection along each arterial. For limited access roadways data are generally tabulated by milepost. A more useful summarization is to aggregate the data to roadway sections bounded by interchanges. Typical data compilations are shown in Tables 16 and 17.

Results of the accident/incident survey may be compiled in a form similar to that shown in Table 18. The results may then be expanded, as applicable, to an annual basis.

6.9.5 Data Collection Costs

The collection costs for acquiring accident data will be affected by the format and accessibility of the existing record keeping procedure. Analysis costs will include the correlation, summarization and graphical presentation of the data. Based upon experience, the costs for accident data collection and analysis should be within the $3000 to $5000 range, unless the agency already has a computerized data retrieval system, in which case costs will be substantially less.

Collection costs for incident data can be excessive if a means of sampling the corridor roadways is not used. In a busy corridor tens of thousands of incidents will occur annually. An attempt to summarize or even thoroughly scan all the individual incident reports would be a large undertaking. Therefore the special 2-week survey should be used.

NYSDOT - INCIDENT STUDY REPORT   date April/ /76   day S M T W T F S

LOCATION: ☐ LIE ☐ NSP ☐ Other(specify)________   DIRECTION: EB WB NB SB

between ________ & ________

(exit no. or description)

Incident occurred on: ☐ left ☐ center ☐ right ☐ shoulder ☐ ramp ☐ other________

TYPE:

☐ Accident	Breakdown		CONDITIONS:
☐ rear end	☐ Flat Tire	No. vehicles involved___	☐ clear
☐ sideswipe	☐ Fire	No. tow trucks req'd___	☐ rain
☐ head on	☐ Mechanical	No. ambulances req'd___	☐ sleet, snow
☐ off the road	☐ Out of Gas	No. injured___	☐ daylight
	☐ Misc.	No. fatalities___	☐ dusk
			☐ darkness

EFFECT ON TRAFFIC FLOW	Same Direction as Incident	Opposite Direction of Incident
	☐ none ☐ minor ☐ major	☐ none ☐ minor ☐ major
Speeds reduced to:	☐ 55 ☐ 45 ☐ 35 ☐ below 30	☐ 55 ☐ 45 ☐ 35 ☐ below 30
No. lanes closed to traffic ____	☐ left ☐ center ☐ right	

Please describe other effects________

DURATION OF INCIDENT:

Time of incident occurrence________ Time obtained from ☐ motorist ☐ radio dispatch ☐ sight

Time discovered or notified________ notification by ☐ radio dispatch ☐ sight ☐ other

Time police arrive________ Depart________

Time tow truck arrives________ Departs________

Time ambulance arrives________ Departs________

Time roadway is cleared________

Time traffic flow returns to normal________ ☐ observed ☐ estimated

Vehicles Towed? ☐ Yes ☐ No

Source: Long Island IMIS Feasiblity Study   1 mph = 1.61 km/h

Figure 11. Example of Incident Report Form

Table 16. Example of Accident Summary (Limited Accessibility)

SECTION	1972 E/B	1972 W/B	1972 TOTAL	1974 TOTAL
New York City Line				
Lakeville Road	30	55	85	23
Community Drive	5	11	16	23
New Hyde Park Road	14	36	50	35
Shelter Rock Road	23	21	44	39
Searingtown Road	26	9	35	32
Willis Avenue	31	58	89	47
Roslyn Road	50	18	68	15
Locust Lane	4	3	7	10
Clen Cove Road	19	34	53	60
Red Ground Road	12	25	37	18
Wheatley Road	8	11	19	11
Old Westbury Road	3	11	14	6
Post Road	13	20	33	7
West Powell Lane	6	5	11	4
Powell Lane	9	9	18	10
Jericho Turnpike	41	35	76	21
Broadway	27	24	51	36
S. Oyster Bay Road	45	33	78	43
Seaford Oyster Bay Exp	15	22	37	52
Manetto Hill Road	28	39	67	17
Northern State Pkwy. Connectors	17	40	57	19
Sunnyside Blvd.	18	14	32	22
Round Swamp Road	24	15	39	30
Route 110	37	17	54	65
Bagatelle Road	39	21	60	36
Route 231	33	20	53	37
Commack Road	44	30	74	34
Sagtikos Parkway	18	15	33	22
Wicks Road	14	10	24	19
Motor Parkway	13	9	22	31
Route 111	15	14	29	16
Veterans Highway	20	18	38	26
Totals	701	702	1403	866

Source: Long Island IMIS Feasibility Study

Table 17. Example of Intersection Accident Summary

Nassau County Intersection Accident Summary

Main Street	Minor Street	Number Of Accidents '72	'73	'74	Road Jurisdiction
Jericho Turnpike	Underhill Boulevard	24	12	27	State
Old Country Road	Post Ave/Merrick Ave	25	24	26	County
Long Island Exp S. Svs Rd	Broadway (Rts 106 & 107)	24	29	25	State
South Oyster Bay Road	Woodbury Road	14	25	22	County
Northern Boulevard	Community Drive	27	15	21	State
Jericho Turnpike	Nassau Boulevard	23	21	21	State
Newbridge Road	West John Street	31	22	20	State
Northern Boulevard	Glen Cove Road	28	37	19	State
Jericho Turnpike	Piquets La/Southwoods Rd	14	31	19	State
Marcus Avenue	New Hyde Park Road	26	33	18	County
Jericho Turnpike	Brush Hollow Road	16	24	18	State
Newbridge Road	Duffy Avenue	-	-	17	State
Old Country Road	Glen Cove Road	-	15	17	County
Herricks Road	Hillside Avenue	-	14	17	State
Jericho Turnpike	Denton Avenue	16	-	17	State
Broadway (Rts 106 & 107)	Columbia Drive	17	10	17	State
Jericho Turnpike	Crossways Park Drive	19	13	16	State
Jericho Turnpike	Robbins Lane	15	15	14	State
Jericho Turnpike	Lafayette Drive	-	-	12	State
Jericho Turnpike	Bruce Street	-	-	11	State
Long Island Exp S. Svs Rd	Jericho Turnpike	27	-	-	State
Hillside Avenue	Willis Avenue	23	-	-	State
Northern Boulevard	Lakeville Road	22	-	-	State
Broadway (Rts 106 & 107)	Nevada Road	19	-	-	State
Long Island Exp S. Svs Rd	Searingtown Road	18	-	-	County
Old Country Road	Zeckendorf Boulevard	18	-	-	County
Jericho Turnpike	Woodbury Road	18	-	-	State
Northern Boulevard	Plandome Road	17	-	-	State
Jericho Turnpike	Mineola Boulevard	17	-	-	State
Long Island Exp S. Svs Rd	Willis Avenue	16	-	-	County
Hillside Avenue	New Hyde Park Road	16	-	-	State
Northern Boulevard	Broadway	15	-	-	State
Northern Boulevard	Jayson Avenue	15	-	-	State
Northern Boulevard	Shelter Rock Road	15	-	-	State
Jericho Turnpike	Willis Avenue	15	-	-	State
Glen Cove Road	Westbury Avenue	15	-	-	County
Northern Boulevard	Underdonk Avenue	14	-	-	State
Hillside Avenue	Lakeville Road	14	-	-	State
Jericho Turnpike	New Hyde Park Road	14	-	-	State
Old Country Road	Newbridge Road (Rt 106)	14	-	-	County

Note: Included in the preceeding table are intersections that accumulated 10 or more total accidents in the given year.

Source: Long Island IMIS Feasibility Study

Table 18. Example of Accident/Incident Study Summary

	LONG ISLAND EXPRESSWAY - WESTBOUND								
Location	No. Of Accidents	Mech. Brkdwns	Flats	Out Of Gas	Misc.	Had No Effect	Had An Effect	Duration (mins)	No. Towed
Veterans Memorial Hwy - Route 111	0	2	0	0	0	0	2	3	0
Route 111 - Motor Parkway	0	1	0	0	0	1	0	-	0
Motor Parkway - Wicks Road	0	4	0	1	1	3	3	38	2
Wicks Road - Sagtikos State Pkwy	0	1	1	0	0	1	1	40	0
Sagtikos State Pkwy - Commack Rd	0	1	0	0	0	1	0	-	0
Commack Road - Route 231	1	6	2	0	0	7	2	45	2
Route 231 - Bagatelle Road	2	8	5	0	0	13	2	81	7
Bagatelle Road - Route 110	0	3	5	0	0	6	2	40	0
Route 110 - Round Swamp Road	0	7	2	1	1	7	4	35	1
Round Swamp Road - Sunnyside Blvd	0	4	2	2	1	7	2	33	0
Sunnyside Blvd - Northern State Pkwy	2	4	2	0	3	3	8	31	3
Northern State Pkwy - Seaford-Oyster Bay Expressway	0	1	1	0	0	2	0	-	0
Seaford-Oyster Bay Expressway - South Oyster Bay Road	0	4	1	1	0	4	2	15	1
South Oyster Bay Road - Northern State Parkway (42)	1	2	0	0	0	1	2	55	2
Broadway - Jericho Turnpike	0	1	1	0	0	0	2	33	0
Jericho Turnpike - Glen Cove Road	3	15	4	2	2	13	13	42	8
Glen Cove Road - Northern State (38)	0	0	1	0	1	1	1	90	0
Northern State Pkwy - Willis Ave	2	5	1	0	2	5	5	30	2
Willis Ave - Searingtown Road	0	2	0	0	0	2	0	-	1
Searingtown Rd - Shelter Rock Rd	0	5	1	0	1	3	4	24	3
Shelter Rock Rd - New Hyde Park Rd	1	9	5	2	2	10	9	21	5
New Hyde Park Rd - Community Drive	0	0	1	1	0	2	0	-	0
Community Drive - City Line	0	4	0	1	0	5	0	-	2
Little Neck Pkwy - Douglaston Pkwy	0	7	0	0	0				

Source: Long Island IMIS Feasibility Study

Since the special study would involve police or tow operator personnel, most of the collection cost is born by other agencies. The analysis of incident data includes the correlation, summarization, and graphical presentation of the data. Based upon experience, the costs for incident data analysis should also be in the $3000 to $5000 range.

6.9.6 Alternatives

It is possible to apply alternative procedures which can greatly reduce the effort on the accident/incident data task. For areas relating to assessment of route quality, a qualitative approach can be used based on general knowledge of the roadways, experience, and perhaps a minimal review of the available data. This assessment is primarily needed for the arterials. Here, for example, the arterials can be rated as "low", "moderate", or "high" as regards the accident frequency.

For the freeways, some quantitative estimate will be required. Here, a procedure may be used which is based on the availability of state-wide accident data for various roadway categories. After a review of available literature, it was concluded that the best single source of fatal and injury accident data is the FHWA document "Fatal and Injury Accident Rates on Federal Aid and Other Highway Systems". This publication is published yearly, and hence the most up-to-date edition should be used. Tables 19 and 20 illustrate typical examples from this document. Although roadway classification systems vary and are subject to change, the methodology user should be capable of selecting an appropriate category to match the roadways in his system. This document provides statistics on fatal and non-fatal injury accidents only. The total accidents (fatal and non-fatal injury + property damage only) may then be estimated as follows:

$$\text{Accident Rate Total} = F \quad ((\text{Accident Rate Fatal}) + (\text{Accident Rate Non-Fatal Injury}))$$

where the expansion factor F is the inverse of the fraction of the total accidents that are in the fatal and non-fatal injury categories. For a data sample based on eight reference sources * the mean fraction of accidents which are in the fatal and non-fatal injury accident category was .34. Therefore, the expansion factor F is 3.0 and as a rule of thumb the total reported accident rate is three times the fatal plus non-fatal injury accident rate.

The requirement for incident data (non-accident) associated with limited access roadways is quite important. These incidents must be addressed if a correct assessment of benefits with respect to system operation is to be developed for those time intervals when lane-blockages affect the roadway. The alternative method is to use a factor which defines the correspondence between all lane blockages, lane blockages due to reported accidents and remaining lane

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* Chicago 1958, Milwaukee 1962-70, California 1963, Florida 1975, Louisiana 1976, Michigan 1975, New York 1970-73

Table 19. Sample Page from FHWA's "Fatal and Injury Accident Rate" Document

1975 URBAN FATAL ACCIDENT RATES BY HIGHWAY SYSTEM AND STATE

(Rates are per 100 million vehicle miles)

TABLE FAR-3

STATE	Non-Inter state	FEDERAL-AID: PRIMARY: Interstate final†	FEDERAL-AID: PRIMARY: Interstate traveled way†	FEDERAL-AID: PRIMARY: Other†	FEDERAL-AID: PRIMARY: Subtotal	FEDERAL-AID: SECONDARY: State†	FEDERAL-AID: SECONDARY: Local‡	FEDERAL-AID: SECONDARY: Subtotal	FEDERAL-AID: URBAN†	FEDERAL-AID: Subtotal	NON-FEDERAL-AID: State†	NON-FEDERAL-AID: Local‡	NON-FEDERAL-AID: Subtotal	TOTAL	Total state systems	Total non-state systems	STATE
ALABAMA	2.49	1.13	2.16	2.50	2.24	2.84	2.95	2.89	2.50	2.33	2.24	2.47	2.46	**2.39**	2.30	2.50	ALABAMA
ALASKA	1.24	----	----	1.37	1.37	4.60	----	4.60	----	1.90	----	.57	.57	**1.24**	1.90	.57	ALASKA
ARIZONA	2.96	2.13	2.72	5.68	3.74	16.28	6.01	7.18	1.90	2.58	16.67	3.91	3.94	**2.91**	2.41	4.20	ARIZONA
ARKANSAS	2.21	1.79	----	2.23	2.09	2.33	.00	1.99	2.28	2.11	2.82	2.23	2.26	**2.15**	2.18	2.08	ARKANSAS
CALIFORNIA	2.66	.70	1.11	1.48	1.11	3.71	4.97	4.66	2.72	1.89	1.55	4.33	4.30	**2.27**	1.85	4.37	CALIFORNIA
COLORADO	2.07	1.23	----	1.80	1.58	3.71	----	3.71	2.96	2.16	2.63	1.35	1.37	**1.93**	2.17	1.35	COLORADO
CONNECTICUT	2.00	1.00	.79	2.30	1.47	----	----	----	2.05	1.75	1.11	2.28	1.86	**1.77**	1.70	2.28	CONNECTICUT
DELAWARE	2.71	.51	-	2.50	1.64	3.06	----	3.06	3.09	2.25	----	2.03	2.03	**2.23**	2.25	2.03	DELAWARE
DIST. OF COL	2.61	.63	1.16	3.36	2.56	----	2.46	2.46	----	2.53	----	2.11	2.11	**2.40**	2.56	2.24	DIST OF COL
FLORIDA	2.11	.86	3.21	3.14	2.26	3.61	1.47	3.19	2.44	2.48	1.68	1.48	1.49	**2.02**	2.47	1.48	FLORIDA
GEORGIA	2.97	1.27	5.65	5.59	3.47	1.48	.00	.73	2.44	3.08	.00	2.43	2.31	**2.66**	3.09	2.30	GEORGIA
HAWAII	2.05	1.17	1.70	1.32	1.32	1.89	4.48	3.33	2.11	1.69	---	2.67	2.67	**1.89**	1.58	2.89	HAWAII
IDAHO	4.08	.65	.00	2.70	1.72	.00	4.35	1.89	2.93	2.49	---	5.69	5.69	**3.79**	2.45	5.65	IDAHO
ILLINOIS	2.71	1.08	1.14	2.98	2.20	3.23	4.40	3.70	3.93	2.61	2.44	2.14	2.18	**2.43**	2.53	2.26	ILLINOIS
INDIANA	1.96	1.43	1.81	1.82	1.64	1.89	2.74	2.09	2.47	2.03	2.00	1.64	1.64	**1.89**	2.01	1.69	INDIANA
IOWA	1.65	.84	1.15	2.65	2.06	----	1.48	1.48	1.80	1.94	1.21	1.25	1.25	**1.58**	1.97	1.27	IOWA
KANSAS	2.01	1.21	5.77	1.26	1.35	5.71	3.16	3.33	.81	1.56	.00	2.38	2.35	**1.92**	1.27	2.50	KANSAS
KENTUCKY	1.99	1.19	1.24	2.66	2.10	2.07	1.10	1.99	2.25	2.09	1.86	1.47	1.55	**1.90**	2.08	1.45	KENTUCKY
LOUISIANA	2.58	1.23	4.41	1.60	1.51	2.45	----	2.45	3.02	2.15	4.28	2.97	3.21	**2.36**	2.24	2.97	LOUISIANA
MAINE	1.82	4.13	.00	2.49	2.73	1.36	----	1.36	4.10	2.60	.31	1.79	1.25	**1.97**	2.04	1.79	MAINE
MARYLAND	2.42	.70	.22	1.89	1.27	2.12	4.17	2.59	.00	1.53	17.14	3.18	3.51	**1.97**	1.51	3.33	MARYLAND
MASSACHUSETTS	3.26	.81	.00	1.91	1.50	2.14	1.80	1.91	8.24	2.53	1.46	8.23	4.17	**2.75**	2.52	3.93	MASSACHUSETTS
MICHIGAN	2.02	1.42	2.47	2.05	1.82	2.08	1.43	1.91	2.19	2.02	1.56	1.40	1.40	**1.92**	2.02	1.40	MICHIGAN
MINNESOTA	1.81	.67	1.23	1.33	1.07	.00	.35	.23	3.06	1.77	2.27	1.24	1.26	**1.63**	1.82	1.18	MINNESOTA
MISSISSIPPI	2.26	.89	3.80	3.23	2.76	.92	.28	.43	1.28	1.97	.00	2.59	2.57	**2.13**	2.16	2.07	MISSISSIPPI
MISSOURI	2.52	1.12	.53	2.14	1.45	2.49	.00	2.46	2.37	1.87	2.35	3.11	3.07	**2.15**	1.88	3.10	MISSOURI
MONTANA	2.43	2.99	---	6.15	5.50	----	----	----	1.61	3.29	---	1.44	1.44	**2.45**	3.29	1.44	MONTANA
NEBRASKA	1.60	1.32	.00	2.76	2.28	6.67	3.03	3.51	2.06	2.30	----	1.14	1.14	**1.58**	2.26	1.20	NEBRASKA
NEVADA	3.28	1.83	.00	2.44	2.19	10.00	7.29	8.33	3.45	3.57	.00	2.73	2.67	**3.16**	3.14	3.18	NEVADA
NEW HAMPSHIRE	2.48	1.40	1.61	1.96	1.74	3.90	.00	3.75	2.29	2.11	1.45	3.71	2.91	**2.36**	2.03	3.68	NEW HAMPSHIRE
NEW JERSEY	2.03	1.34	3.55	1.98	1.86	----	----	----	2.35	2.11	1.10	1.88	1.71	**1.96**	1.99	1.88	NEW JERSEY
NEW MEXICO	3.24	1.15	11.11	3.97	3.37	12.50	.00	11.65	2.17	3.04	2.04	2.77	2.73	**2.96**	3.03	2.75	NEW MEXICO
NEW YORK	2.66	1.15	.69	.73	.88	1.34	3.77	2.41	4.35	1.73	.47	3.65	3.56	**2.41**	1.63	3.65	NEW YORK
NORTH CAROLINA	1.89	.71	1.95	1.90	1.61	1.88	.00	1.87	1.92	1.74	1.89	1.87	1.87	**1.78**	1.75	1.86	NORTH CAROLINA
NORTH DAKOTA	2.32	2.04	.00	3.40	3.15	.00	4.17	3.70	.72	2.10	.00	2.77	2.75	**2.31**	2.01	2.88	NORTH DAKOTA
OHIO	2.85	.74	4.01	3.41	1.91	3.21	3.88	3.59	4.39	2.54	.27	1.88	1.83	**2.30**	2.33	2.26	OHIO
OKLAHOMA	1.72	2.25	----	2.81	2.48	3.85	2.27	3.13	.86	1.42	.00	2.54	2.52	**1.79**	1.40	2.53	OKLAHOMA
OREGON	3.38	1.46	2.48	3.48	2.57	6.99	1.51	2.34	----	2.52	1.67	4.05	3.96	**2.95**	2.76	3.19	OREGON
PENNSYLVANIA	3.03	1.39	2.99	2.91	2.62	3.74	3.64	3.74	3.45	3.11	3.65	1.89	2.42	**2.92**	3.16	1.92	PENNSYLVANIA
RHODE ISLAND	1.84	.78	2.30	1.57	1.31	2.42	2.87	2.55	2.83	1.80	1.49	.79	.84	**1.63**	1.74	1.23	RHODE ISLAND
SOUTH CAROLINA	2.61	1.29	7.14	1.95	1.88	3.81	.00	3.75	----	2.34	3.11	2.29	2.95	**2.50**	2.51	2.16	SOUTH CAROLINA
SOUTH DAKOTA	2.06	2.50	20.00	2.89	3.09	----	----	----	.84	1.97	.00	2.36	2.34	**2.08**	1.96	2.36	SOUTH DAKOTA
TENNESSEE	2.10	1.23	1.29	2.43	1.95	4.36	1.40	3.07	.63	1.77	3.00	2.17	2.20	**1.97**	1.81	2.15	TENNESSEE
TEXAS	3.08	1.73	4.53	2.59	2.20	2.88	----	2.88	3.00	2.48	2.06	3.52	3.49	**2.79**	2.48	3.52	TEXAS
UTAH	2.61	.91	3.70	1.86	1.12	2.82	2.56	2.76	2.49	2.00	.00	3.17	3.12	**2.18**	1.99	3.14	UTAH
VERMONT	3.40	.00	.00	1.89	1.57	.00	7.69	3.45	10.71	2.39	.00	4.41	4.37	**3.14**	2.19	4.58	VERMONT
VIRGINIA	2.05	1.20	1.37	2.47	1.91	1.94	2.40	2.04	3.43	2.40	.59	1.04	1.03	**1.92**	2.37	1.08	VIRGINIA
WASHINGTON	2.75	.75	2.70	1.81	1.22	3.41	2.94	3.15	3.08	1.94	.00	2.91	2.90	**2.25**	1.82	2.91	WASHINGTON
WEST VIRGINIA	2.47	.00	2.67	3.68	2.72	3.16	4.05	3.33	2.41	2.82	.66	.99	.95	**2.17**	2.63	1.20	WEST VIRGINIA
WISCONSIN	1.73	.53	1.65	2.06	1.67	.00	1.33	1.23	1.33	1.51	.00	1.82	1.82	**1.63**	1.52	1.78	WISCONSIN
WYOMING	2.17	2.50	.00	5.32	4.23	----	----	----	2.48	3.13	.00	1.16	1.15	**2.19**	3.12	1.16	WYOMING
TOTAL	**2.47**	**1.08**	**2.06**	**2.25**	**1.77**	**2.86**	**2.82**	**2.85**	**2.70**	**2.17**	**2.02**	**2.44**	**2.40**	**2.24**	**2.14**	**2.47**	**TOTAL**
REVISED TOTALS																	
1974	2.52	1.18	2.19	2.50	2.00	2.96	2.84	2.91	2.44	2.22	2.22	2.51	2.48	2.31	2.19	2.54	
1973	2.75	1.61	2.43	2.86	2.39	3.49	2.75	3.15	2.94	2.61	2.55	2.53	2.53	2.58	2.59	2.56	
1972	2.84	1.66	2.41	3.03	2.49	3.39	2.98	3.20	2.90	2.67	3.06	2.62	2.66	2.66	2.66	2.66	
1971	2.87	1.70	2.59	2.72	2.36	3.04	3.47	3.24	---	2.54	2.55	2.91	2.88	2.69	2.45	2.97	
1970	3.07	1.67	2.50	2.85	2.42	3.44	3.33	3.39	----	2.61	2.76	3.19	3.15	2.87	2.56	3.21	
1969	3.26	1.97	2.87	3.17	2.76	3.40	3.69	3.54	----	2.91	2.65	3.35	3.28	3.09	2.81	3.38	
1968	3.38	1.91	2.78	3.35	2.85	3.46	4.34	3.87	----	3.05	2.65	3.43	3.36	3.20	2.90	3.52	
1967	3.44	1.81	2.95	3.40	2.89	3.52	3.93	3.71	----	3.05	2.94	3.52	3.46	3.25	2.96	3.55	

†State system
‡Non-state system

Table 20. Second Sample Page from FHWA's "Fatal and Injury Accident Rate" Document

1975 URBAN NON-FATAL INJURY ACCIDENT RATES BY HIGHWAY SYSTEM AND STATE

(Rates are per 100 million vehicle miles)

TABLE IAR-3

STATE	Non-Interstate	FEDERAL-AID									NON-FEDERAL-AID			TOTAL	Total state systems	Total non-state systems	STATE
		PRIMARY				SECONDARY			URBAN†	Subtotal	State†	Local‡	Subtotal				
		Interstate final†	Interstate traveled way†	Other†	Subtotal	State†	Local‡	Subtotal									
ALABAMA	108.72	31.00	77.37	92.54	80.83	100.77	116.22	107.98	116.10	86.70	89.78	124.87	122.43	**103.21**	85.41	124.35	ALABAMA
ALASKA	115.30	----	----	158.22	158.22	345.98	----	345.98	----	189.33	----	41.56	41.56	**115.30**	189.33	41.56	ALASKA
ARIZONA	214.65	116.49	125.00	252.58	176.13	541.86	505.11	509.31	137.33	166.97	933.33	336.84	338.61	**208.11**	149.29	360.64	ARIZONA
ARKANSAS	219.35	58.27	----	185.29	144.80	211.99	5.32	182.24	251.50	173.49	361.97	251.59	257.51	**196.76**	181.98	234.42	ARKANSAS
CALIFORNIA	175.94	26.62	43.41	74.32	50.98	175.71	298.41	268.10	167.13	106.56	290.70	357.33	356.75	**146.13**	104.26	353.38	CALIFORNIA
COLORADO	187.76	75.98	----	183.18	141.47	202.86	----	202.86	204.37	165.93	100.00	177.93	176.74	**169.15**	165.50	177.93	COLORADO
CONNECTICUT	209.42	57.07	83.93	198.00	112.10	----	----	----	234.95	171.11	97.68	238.70	187.85	**174.22**	165.31	240.18	CONNECTICUT
DELAWARE	202.71	56.85	----	164.49	118.14	146.72	----	146.72	181.90	140.33	----	416.75	416.75	**170.68**	140.33	416.75	DELAWARE
DIST. OF COL.	77.23	11.91	111.63	79.38	69.16	----	117.37	117.37	----	82.04	----	44.47	44.47	**70.47**	69.16	71.82	DIST. OF COL.
FLORIDA	171.82	63.74	147.52	194.02	136.57	214.84	148.35	201.95	201.84	177.66	140.19	148.38	147.85	**163.58**	176.33	148.38	FLORIDA
GEORGIA	113.10	33.87	172.17	172.37	104.34	207.19	112.32	159.06	89.51	110.30	81.49	89.43	89.04	**98.63**	108.43	90.60	GEORGIA
HAWAII	234.99	101.86	138.07	162.48	137.44	288.68	225.37	253.33	277.29	186.62	----	306.38	306.38	**211.03**	185.20	296.56	HAWAII
IDAHO	238.43	27.92	170.00	233.51	140.97	243.33	260.87	250.94	257.48	219.65	----	222.09	222.09	**220.64**	218.76	223.26	IDAHO
ILLINOIS	267.46	76.02	166.44	256.31	183.77	226.74	222.49	225.02	379.76	219.01	315.57	246.52	256.42	**234.51**	228.08	245.23	ILLINOIS
INDIANA	180.51	14.01	328.62	142.87	90.62	133.45	306.40	172.93	180.50	136.55	95.00	192.78	191.34	**156.30**	131.47	198.11	INDIANA
IOWA	197.18	47.07	126.44	242.99	181.78	----	263.64	263.64	208.99	197.07	70.91	175.64	171.50	**183.72**	182.17	184.94	IOWA
KANSAS	298.34	65.95	432.69	227.85	178.40	351.43	245.65	252.50	238.55	201.24	35.29	358.50	354.53	**270.18**	191.60	340.88	KANSAS
KENTUCKY	129.68	37.39	51.06	145.12	105.13	145.26	150.55	145.67	124.08	115.19	91.98	136.64	126.96	**119.37**	112.17	137.12	KENTUCKY
LOUISIANA	323.99	127.50	148.53	273.86	212.88	246.36	----	246.36	369.97	268.24	459.87	370.01	386.55	**291.60**	276.54	370.01	LOUISIANA
MAINE	170.14	77.69	175.00	242.72	210.77	180.45	----	180.45	129.51	194.21	130.00	129.93	129.95	**164.18**	178.65	129.93	MAINE
MARYLAND	240.10	47.55	94.83	149.51	101.69	170.90	612.90	271.93	132.16	138.77	1004.29	353.78	369.46	**189.64**	120.62	392.90	MARYLAND
MASSACHUSETTS	229.09	47.90	222.45	165.61	124.05	90.17	135.20	120.84	501.17	177.05	137.62	501.13	283.50	**191.04**	178.65	256.52	MASSACHUSETTS
MICHIGAN	215.02	45.41	177.38	159.50	111.79	241.15	237.14	240.08	231.86	178.13	128.13	236.89	235.68	**187.55**	177.88	236.89	MICHIGAN
MINNESOTA	154.48	50.51	67.83	90.32	73.28	113.14	120.07	117.84	236.04	135.87	95.45	143.64	143.12	**137.95**	136.16	142.06	MINNESOTA
MISSISSIPPI	73.23	18.22	86.55	76.35	64.07	44.04	82.54	73.49	79.34	70.26	11.11	61.84	61.48	**67.91**	68.66	66.46	MISSISSIPPI
MISSOURI	112.92	35.57	8.51	86.96	52.42	123.48	125.00	123.50	123.49	84.18	44.60	123.47	118.91	**92.29**	83.46	123.48	MISSOURI
MONTANA	154.51	100.00	----	246.15	216.21	----	----	----	118.89	160.71	----	141.12	141.12	**151.88**	160.71	141.12	MONTANA
NEBRASKA	205.69	63.68	266.67	312.83	235.23	266.67	307.07	301.75	319.91	260.28	----	153.63	153.63	**194.17**	257.53	158.69	NEBRASKA
NEVADA	223.14	108.54	110.00	184.45	158.17	138.33	229.17	194.23	217.70	187.01	190.48	247.49	246.16	**213.70**	183.10	245.69	NEVADA
NEW HAMPSHIRE	196.52	29.91	12.90	266.20	161.67	207.79	.00	200.00	163.83	164.98	38.65	299.47	207.02	**177.84**	148.20	297.11	NEW HAMPSHIRE
NEW JERSEY	241.59	49.58	260.94	251.90	181.58	----	----	----	245.15	214.42	96.68	268.93	233.28	**221.81**	200.58	268.93	NEW JERSEY
NEW MEXICO	224.54	66.22	288.89	241.28	178.55	378.13	57.14	356.31	228.14	209.89	175.51	183.39	182.97	**203.69**	209.69	182.38	NEW MEXICO
NEW YORK	330.95	98.50	82.34	82.29	88.18	159.37	569.28	340.85	490.09	193.12	47.99	474.30	462.90	**293.43**	176.24	480.14	NEW YORK
NORTH CAROLINA	162.29	26.35	162.30	162.31	127.92	162.29	161.90	162.29	162.25	144.45	162.30	162.28	162.29	**149.55**	145.61	162.28	NORTH CAROLINA
NORTH DAKOTA	244.43	36.73	33.33	249.06	214.20	66.67	245.83	225.93	213.04	214.19	.00	275.78	273.88	**233.26**	212.21	273.48	NORTH DAKOTA
OHIO	263.00	48.32	264.47	292.86	153.47	256.89	339.50	303.79	387.84	211.40	34.86	205.04	199.56	**207.47**	192.50	230.32	OHIO
OKLAHOMA	81.72	31.53	----	75.23	49.52	111.54	38.64	78.13	54.04	52.89	119.35	119.58	119.58	**75.40**	53.25	118.65	OKLAHOMA
OREGON	322.35	64.45	299.17	241.39	167.89	767.83	88.07	191.59	----	173.72	31.67	493.57	476.41	**263.72**	193.09	356.27	OREGON
PENNSYLVANIA	186.01	86.50	137.80	128.29	120.67	200.82	200.91	200.82	204.82	162.96	149.68	253.51	222.63	**179.13**	161.48	252.61	PENNSYLVANIA
RHODE ISLAND	193.53	13.30	139.08	133.26	89.31	208.06	221.53	212.06	200.60	131.44	311.94	281.28	283.75	**158.62**	129.61	268.45	RHODE ISLAND
SOUTH CAROLINA	119.19	28.48	217.86	102.85	92.11	118.12	00	116.22	----	98.07	140.07	193.98	150.74	**111.25**	107.27	182.97	SOUTH CAROLINA
SOUTH DAKOTA	208.14	72.50	.00	256.91	232.58	----	----	----	115.17	173.88	150.00	272.39	271.57	**202.77**	173.81	272.39	SOUTH DAKOTA
TENNESSEE	121.75	38.86	85.84	127.29	93.44	219.64	172.90	199.18	80.13	96.94	160.94	122.47	123.67	**109.25**	96.78	123.92	TENNESSEE
TEXAS	175.92	67.46	189.33	115.23	93.10	105.28	----	105.28	163.72	114.74	109.14	241.22	238.12	**152.13**	114.68	241.22	TEXAS
UTAH	246.46	55.90	162.96	151.63	73.48	248.59	184.62	234.81	231.70	173.86	300.00	330.36	329.87	**198.19**	174.13	322.25	UTAH
VERMONT	168.69	69.57	233.33	110.61	108.15	131.25	546.15	317.24	453.57	150.00	700.00	174.89	179.48	**161.16**	138.90	195.00	VERMONT
VIRGINIA	150.66	38.34	115.07	182.71	124.20	127.00	96.00	120.41	211.57	151.67	62.13	101.59	100.37	**133.95**	150.88	101.46	VIRGINIA
WASHINGTON	285.31	56.32	222.97	181.13	108.77	224.94	322.11	278.73	335.82	183.79	900.00	322.17	322.83	**228.12**	166.80	322.16	WASHINGTON
WEST VIRGINIA	170.86	42.82	225.33	185.02	154.71	143.04	198.65	153.59	167.47	155.00	150.66	156.42	155.67	**155.23**	153.29	159.30	WEST VIRGINIA
WISCONSIN	168.74	47.29	190.12	158.94	133.65	142.55	165.84	164.15	142.34	139.23	350.00	188.83	188.88	**159.07**	137.24	186.61	WISCONSIN
WYOMING	201.30	87.50	50.00	211.70	167.61	----	----	----	314.88	260.42	400.00	121.97	122.77	**195.08**	260.78	121.97	WYOMING
TOTAL	**200.85**	**51.73**	**126.63**	**154.94**	**111.89**	**181.43**	**252.61**	**205.75**	**214.48**	**153.98**	**152.91**	**234.64**	**227.55**	**176.72**	**150.71**	**235.98**	**TOTAL**
REVISED TOTALS																	
1974	196.54	50.30	137.67	164.83	121.22	179.69	209.08	192.09	209.61	151.26	159.92	224.98	219.71	173.74	148.27	223.20	
1973	198.74	62.45	134.82	164.44	126.29	169.38	200.71	183.80	225.16	152.28	165.86	229.29	224.08	177.97	149.13	225.59	
1972	200.12	64.61	128.20	166.86	127.29	178.16	225.08	199.49	217.24	151.67	189.99	224.66	221.87	179.91	148.04	224.70	
1971	208.81	64.42	133.29	154.70	121.81	163.53	220.77	189.40	----	135.50	166.21	256.77	248.06	187.50	130.19	252.98	
1970	225.01	65.31	139.96	169.18	131.53	175.25	248.43	208.80	----	146.77	178.58	272.67	263.79	202.35	140.26	270.26	
1969	232.65	67.99	142.04	180.28	140.34	176.87	274.14	221.80	----	155.98	184.11	278.92	269.97	210.39	147.91	278.46	
1968	232.00	65.90	144.36	189.85	147.00	177.02	291.42	229.90	----	163.02	183.93	269.17	261.32	211.32	153.41	271.23	
1967	240.94	64.20	143.27	202.72	155.18	184.36	248.58	214.24	----	166.52	177.10	284.30	274.31	220.36	160.20	281.09	

†State system
‡Non-state system

blockages due to incidents. Two sources of data (Gulf Freeway* and Long Island IMIS study) indicate that the rate of lane-blocking incident occurrence is approximately equal to the rate of reported lane-blocking accidents. Thus, the total rate at which lane blockages occur may be estimated to be twice the rate of occurrence of reported accidents.

Use of the above alternatives for accident/incident data is not expected to alter the outcome of the feasibility study, since related benefits are only a portion of the total, and the estimates used should be reasonable approximations of the true values.

6.10 PLANNED HIGHWAY CONSTRUCTION

Improvements to the freeways and arterials will affect traffic flow. It is therefore necessary to evaluate the impact of the improvements upon traffic distribution and capacity.

6.10.1 Uses of the Data

The planned highway construction data and analysis provides input for Methodology Uses A and F. Thus, the data become involved in:

- Capacity Analysis and Excess Capacity Evaluation

- Alternate Route Analysis

6.10.2 Data Requirements and Analysis

Data are required showing the locations, purpose, and estimated results of highway improvements scheduled for completion before or soon after implementation of the proposed IMIS project.

The basic data required are listings of the planned major highway improvements showing route name, project limits, description of construction, and construction schedule. Table 21 represents a sample format for documenting this type of data.

The analysis process will involve appraising the effects of each project upon available capacity and future traffic volumes in the highway segments involved.

6.10.3 Data Collection

The data can be acquired through the appropriate division of the highway agency that has planned and will construct the improvement.

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*Goolsby, M. E., "Influence of Incidents on Freeway Quality of Service", Highway Research Record Number 349, 1971.

Table 21. Example of Planned Highway Construction Summary

PLANNED CONSTRUCTION BY NEW YORK STATE DEPARTMENT OF TRANSPORTATION

P.I.N.	Project Name	Description	Year Construction Start
		NEW CONSTRUCTION	
* 0227.76	Huntington TOPICS (Pinelawn Road Ramp)	Will provide for an additional exit ramp for the westbound Long Island Expressway prior to Pinelawn Road and an additional entrance ramp from Pinelawn Road to the eastbound Long Island Expressway.	77-78
* TRN2.72	Long Island Expressway Service Rds	Will provide for continuation of some Long Island Expsy Service Road sections in order to eliminate two way Service Road segments.	78-79
		MAJOR RECONSTRUCTION PROJECTS	
0025.02	Interborough Parkway - Penn Avenue to Grand Central Parkway	Will provide for reconstruction of the Interborough Pkwy on existing right of way with no additional through lanes added.	80-81
0054.03	Route 111 - Route 495 to Route 454	Will provide for the reconstruction of Route 111 as a four lane boulevard with substantially improved geometrics.	81-82
0327.37	Route 25A - Flushing River Bridge Replacement	Will provide for the replacement of the present drawbridge with a high bridge and upgrading of the existing approaches. No additional lanes will be added.	76-77
* 0523.00	Northern State Parkway/ Meadowbrook Parkway Interchange	Will provide for the upgrading of the Northern State Pkwy/ Meadowbrook State Parkway Interchange geometrics. A significant increase in the interchange's capacity could be expected.	82-83
		SAFETY PROJECTS	
0021.21	Belt Parkway (Whitestone Expsy - Brooklyn-Queens Expressway)	Safety Projects generally include sign modifications, guide rail and median barrier upgrading, impact attenuation devices, removal or protection of fixed roadside objects, and other safety improvements. Alterations to the roadway will not be included although small modifications to ramp geometry might be.	78-79

* indicates those projects which have major significance and should be coordinated with IMIS.

Source: Long Island IMIS Feasibility Study

6.10.4 Data Collection Costs

Collection costs for these data involve the acquisition of highway planning schedules and the preparation of a tabulation of those improvements on the corridor roadways. Also involved is the assessment of the impact these proposals have on traffic distribution and roadway capacity. It is estimated the costs for this task should not exceed $1000.

6.10.5 Alternatives

There are basically no alternatives proposed for this data category.

6.11 PLANNED LAND DEVELOPMENTS

Increases in traffic resulting from the normal residential and commercial developments of an area are generally accounted for in the normal projections of annual increases in traffic volumes. However, a major development, such as regional shopping center or major industrial park creating significant peak hour traffic demands, can increase the usefulness and benefits to be derived from IMIS.

6.11.1 Uses of the Data

The Planned Land Use data and analysis provides input to Methodology Use A. Thus, the data become involved in:

- Alternate Route Analysis

6.11.2 Data Requirements and Analysis

The basic data required are listings of proposed major land use development showing location, type of project, estimated trip generation, and realistic construction schedule.

Appraisal of the impact of each development on peak hour traffic demands at the key locations affected, will indicate the degree of consideration to give to this subject. Although the information available may be insufficient for quantitative analysis, a large land development will certainly increase, rather than diminish, the value of IMIS.

6.11.3 Data Collection

Primary sources for such data are the planning and highway agencies. Also, the developer may have traffic studies available.

6.11.4 Data Collection Costs

It is estimated that the collection and analysis costs for these data should not exceed $1000.

6.11.5 Alternatives

There are basically no alternatives proposed for this data category.

CHAPTER 7

SUPPLEMENTAL ANALYSES

7.1 INTRODUCTION

7.1.1 Objectives

- To develop the basic operating characteristics of the corridor in a form suitable for performing the remaining tasks.
- To develop the accident/incident characteristics of the corridor in a form suitable for performing the analyses of the remaining tasks.
- To perform the additional traffic engineering analyses required, including capacity analysis and origin-destination analysis.

7.1.2 Inputs

- Freeway mainline and ramp counts, surface street counts including turning fractions and master station counts.
- Accident/incident data including location, rate, duration and response times.
- Intersection geometrics and signal timing patterns.
- Network travel times.
- Maps of corridor roadways.

7.1.3 Outputs

- Control probability model coefficients (mean, standard deviation)
- Accident/incident rates for limited access roadway expressed on a lane-mile basis.
- Capacity for each corridor roadway.
- Origin-destination pattern.
- Trip length distribution.

7.2 OVERVIEW

The assembly of the data base and the basic data reduction and analyses were described in the previous chapter (Chapter 6). In this chapter, a set of supplemental analyses are performed to provide the remaining necessary inputs, in appropriate forms, for the remaining portions of the study. Five subject areas are treated, corresponding to the five outputs listed above.

The control probability model will be discussed briefly in Section 7.3 and in more detail in Chapter 10. At present, the objective is to develop the coefficients for the model, which are simply the mean and standard deviation of a set of peak hour volume data points.

Accident and incident rates will be required in a specific form for later use in the benefit determination. In this chapter, the previously compiled data for the limited access facilities are converted to the required form.

The capacity analysis, which includes a determination of available capacity (i.e., difference between present demand and capacity), will be required for the later alternate route analysis and estimation of diversion capability. Standard traffic engineering capacity analysis techniques are applicable here.

In the event that the origin-destination model is to be used to determine the ramp-to-ramp O-D patterns, the model is described and a sample problem is included in a referenced appendix.

Trip length distribution (also on a ramp-to-ramp basis) is determined from the origin-destination data. Of specific interest is the median trip length (i.e. distance travelled on facility by 50 percent of the motorists). The specific procedure and a sample problem are contained in a referenced appendix.

7.3 CONTROL PROBABILITY MODEL COEFFICIENTS

IMIS is designed to respond dynamically to varying traffic conditions in the corridor. For example, if a problem exists on one freeway, and the system detects available capacity on an adjacent freeway, a diversion control will be instituted. Available capacity is not a constant, but varies in accordance with normal traffic fluctuations. On any given day, a facility may or may not have available capacity. Thus, a "static" measure of capacity in inadequate for use in evaluating system control potential, since it does not properly describe the dynamic environment. Therefore, an alternate approach is needed. The approach developed for IMIS makes use of a "control probability model", which attempts to quantify (within the limits of probabalistic modelling), the frequency with which capacity is expected to be available for IMIS control. Using sets of volume data, the model establishes the level of traffic variability, which leads eventually to a probability that diversion control can be exercised.

Appendix C further discussed the concept of the control probability model and the derivation of the probability factors for a corridor-specific data set. In Chapter 10, a set of typical control probability factors will be given. In the present

chapter, the task is to develop the coefficients of the model, which are the mean and standard deviation of the volume data set. The equations for computing these parameters are given in Appendix C. The data set required to generate the parameters is composed of hourly count data taken during peak demand periods, converted to a typical peak hour basis (using daily and monthly conversion factors). The data set should consist of at least 30 to 40 data points, with a minimum of 10 data points collected at a single location on a given roadway. Typically, permanent count stations are an excellent source for these data.

7.4 ACCIDENT/INCIDENT RATES

The intent of this subtask is to catalog the accident/incident characteristics for each roadway in the corridor. The representation of this data takes different forms depending upon whether the roadway is an arterial or a limited-access freeway. For arterials the important characteristic is the identification of high accident locations. This will already be available from Chapter 6 (e.g. see Table 17). For freeways the important characteristic is the development of accident and incident rates. The dimensions of these rates as used in the feasibility study are accident or incidents per lane mile per hour of operation. The usefulness of a parameter with these dimensions is that estimates of yearly occurrences can be obtained for any particular time period (AM-peak, PM-peak, off-peak) or roadway segment by a multiplication of the length of the time period, the lane-miles of the roadway segment and the number of days and weeks in a year. To obtain this rate multiply the rate per 100 million vehicle miles (161 million vh. km.) by the typical one-way AADT adjusted to a per lane value. This result is multiplied by the fraction of the AADT which occurs during the time period. This fraction can be obtained from the weekday hourly volume distribution curves (Figure 6). Finally the entire result is divided by 10^8. A single value applicable to all freeway segments in the corridor is acceptable. As was discussed in Chapter 6, when accident statistics are not readily available and the collection of these statistics is considered beyond the scope of the study, alternative values based on statistics contained in the FHWA document "Fatal and Injury Accident Rates on Federal Aid and Other Highway Systems" should be used. Similarly, for incident data the alternative described in Chapter 6 may be used (i.e. incident rates set approximately equal to accident rates).

7.5 CAPACITY ANALYSIS

Each arterial parallel route, arterial connector, and freeway connector must be analyzed to determine the capacity, and the available capacity for the various levels of service. Available capacities are determined by the difference between capacity values and the current use volumes for the time periods studied.

The time periods selected for analysis must include a peak period which contains the peak traffic volumes, travel times, and congestion or capacity restrictions of the total corridor. The data collection process should provide the information necessary to make this selection. Typical peak hours are 7:30 to 8:30 AM and 4:00 to 5:00 PM. Also a typical midday hour should be selected for analysis from the data.

7.5.1 Capacity Analysis of Alternate Parallel Routes

The capacity analysis of routes, parallel to primary freeway facilities is of critical importance because the excess capacity of the parallel routes is a major factor in the selection of alternate routes.

The available capacity of an arterial route segment is limited by the capacity of the signalized intersections along the route. The thru capacities of each route are calculated by using the number of lanes, the green time (split or green to cycle length ratio), and the headways for levels of service C, D, and E. Levels of service C, D and E represent volumes of 1200, 1350, and 1500 vehicles/hour of green/lane or average headways of 3.0, 2.6, and 2.4 seconds/ vehicle respectively. These values are derived from the Highway Capacity Manual (1965), Public Roads Volume 34 (1967), and Intersection Capacity Measurement Through Critical Movement Summations, Planning Tool by H. B. McInerney & S. G. Petersen, ITE Magazine, January 1971.

An example of the calculations are presented in tabular form in Table 22.

The table shows the available capacity restrictions during the PM peak hour along the route. By locating and quantifying these restrictions, it is then possible to determine traffic engineering improvements that will increase the thru capacity on the arterial route. Examples of traffic engineering improvements are improved traffic signal equipment and operation. Table 22 shows that 5 of the 9 intersections operated with 50 percent or less of the green time assigned to the artery. Analysis of the intersection volumes showed that the thru movements along the route can be assigned at least 50 percent and quite often 60 to 70 percent of the total cycle time. Other typical traffic engineering improvements which should be considered are parking restrictions to increase capacity, pavement markings to provide left-turn lanes, and channelization.

By utilizing this method of capacity analysis, easy identification and quantification of problems can be performed with limited data in comparison to manual methods which require substantially more data and unnecessary refinements.

7.5.2 Capacity Analysis of Arterial Connector Routes

The thru capacity and available capacity of arterial routes which provide the connections between the freeways and parallel arterial routes can be found utilizing the same procedures as presented in the preceding discussion. This analysis provides basic input in comparing potential connector routes.

In addition to the thru capacity, analysis must be made at the connections with the freeways and arterials. This analysis will include turning movements, stop and yield signs, capacity of ramps, and the capability of the freeways to handle the ramp merge.

Table 22. Example of Available Capacity Computation

Location	No. Lanes LT	T	RT	Thru Split	Thru Capacity VPH Levels of Service C	D	E	Thru Volumes Ave. Day PM Peak	Ave Hr	Available Capacity PM Peak Hr. Levels of Service C	D	E	Ave. Hr. C	D	E
JERICHO TURNPIKE EASTBOUND FROM 25B MERGE TO CROSSWAYS PARK DRIVE															
25B Merge	1	2			2400	2700	3000	1170	440	1230	1530	1830	1960	2260	2560
Post Road	1	3	1	.50	1800	2020	2250	2020	840	(220)	0	230	960	1180	1410
Brush Hollow Rd.	1	3	1	.27	970	1090	1210	1560	800	(590)	(470)	(350)	170	290	410
Merry Lane	1	3		.70	2520	2830	3150	1140	720	1380	1690	2010	1800	2110	2430
Robbins Lane		2		.38	910	1030	1140	1040	740	(130)	(10)	90	170	290	400
Underhill Blvd.		2		.60	720	810	900	870	790	(150)	(60)	30	(70)	20	110
S. Oyster Bay Rd.	1	2		.45	1080	1210	1350	1300	770	(220)	(90)	50	310	440	580
Woodland Gate	1	2		.70	1680	1890	2100	1510	960	170	380	590	720	930	1140
Crossways Park Dr.		2		.50	1200	1350	1500	970	630	230	380	530	570	720	870
JERICHO TURNPIKE WESTBOUND FROM CROSSWAYS PARK DRIVE TO 25B MERGE															
Crossways Park Dr.	1	2		.50	1200	1350	1500								
Woodland Gate	1	2		.70	1680	1890	2100	1040	940	640	850	1060	740	950	1160
S. Oyster Bay Rd	1	2		.45	1080	1210	1350	870	820	210	340	480	260	390	530
Underhill Blvd.		2		.45	1080	1210	1350	820	910	260	390	530	170	300	440
Robbins Lane		2		.50	900	1010	1120	1120	660	(220)	(110)	0	240	350	460
Merry Lane	1	3		.70	2520	2830	3150	1420	790	1100	1410	1730	1730	2040	2360
South Service Rd		3		.55	1980	2230	2480								
Brush Hollow Rd.	1	3		.41	1470	1660	1840	770	560	700	890	1070	910	1100	1280
Post Road	1	3	1	.65	2340	2630	2920	1370	650	970	1260	1550	1690	1980	2270
25B Merge	1	2			2400	2700	3000	970	630	1430	1730	2030	1770	2070	2370

Source: Long Island IMIS Feasibility Study

The capacity of left turning movements at signalized intersections is dependent upon the frequency of safe gaps through which to turn during the green period. The capacity may be determined by field observation, or estimated based on magnitude of the opposing volume.

The capacity of stop and yield signs is also dependent upon the frequency of safe gaps through which to complete the necessary movement. Either of the alternatives noted above for left turning movements may be used capacity determination.

Ramp thruput capacity should be determined for the appropriate levels of service with adjustments made where applicable for physical conditions.

The capability of the freeways to accommodate the downstream ramp merge must be determined. Truck percentages, geometrics, and internal weaving section traffic volumes must be considered.

Level of service E should be selected for analysis. This provides available capacity information at the level of service providing the greatest number of vehicles that can be accommodated.

Examples of the presentation of the available capacities for a connector route are shown in Table 23. An important aid in analyzing all the potential diversion schemes is a sketch of the routes and the diversion schemes as illustrated by Figure 12.

7.5.3 Capacity Analysis for Freeway to Freeway Connections

Capacities of the ramps and the freeway ramp merge are calculated utilizing the procedures set forth in the Highway Capacity Manual, HRB, SR87, 1965.

A basic ramp thruput capacity of 1800 VPH per lane can be used, with adjustments for truck percentages and physical characteristics.

The amount of ramp traffic that can be merged onto the freeway is equal to the capacity of the freeway downstream, minus the freeway volume upstream. Available ramp merge capacity is the difference between the total that can merge as determined above minus existing ramp traffic.

Examples of this analysis are presented in Table 24. Note that in many instances the ramp capacities are the controlling factor in midday hours with the ramp to freeway merge capacity being critical during the peak hour.

7.6 ORIGIN - DESTINATION PATTERN ANALYSIS WITH ANALYTICAL MODEL

The purpose of this analysis is to develop an estimate of the origin-destination pattern for the major limited access facility or facilities in the

Table 23. Example of Arterial Connector Capactiy Computation

Location	No. Lanes			Thru Split	Thru Capacity VPH Levels of Service			Thru Volumes Ave. Day		Available Capacity PM Peak Hr. Levels of Service			Available Capacity Ave. Hr. Levels of Service		
	LT	T	RT		C	D	E	PM Peak	Ave Hr	C	D	E	C	D	E
NEW HYDE PARK ROAD SOUTHBOUND FROM LONG ISLAND EXPRESSWAY NORTH SERVICE ROAD TO HILLSIDE AVENUE															
LIE N. Service Rd.		1E/S		.470	560	630	700	210	170	350	420	490	390	460	530
LIE S. Service Rd.		2		.530	1270	1430	1590	210	170	1060	1220	1380	1100	1260	1420
Lake Success Quad		2		.700	1680	1890	2100	800	770	880	1090	1300	910	1120	1330
Marcus Avenue	1	2		.485	1160	1310	1450	1020	790	140	290	430	370	520	660
Union Turnpike	1	2		.560	1340	1510	1680	860	760	480	650	820	580	750	920
Hillside Avenue	1	2		.400	960	1080	1200	700	470	260	380	500	490	610	730
NEW HYDE PARK ROAD NORTHBOUND FROM HILLSIDE AVENUE TO LONG ISLAND EXPRESSWAY NORTH SERVICE ROAD															
Hillside Avenue	1	2		.400	960	1030	1200	440	450	520	640	760	510	630	750
Union Turnpike		2		.560	1340	1510	1680	430	780	910	1080	1250	560	730	900
Marcus Avenue	1	2		.485	1160	1310	1450	640	620	520	670	810	540	690	830
Lake Success Quad		2		.700	1680	1890	2100	1150	800	530	740	950	880	1090	1300
LIE South Service Rd		2		.530	1270	1430	1590	1030	630	240	400	560	640	800	960
LIE North Service Rd		2		.530	1270	1430	1590	620	510	650	810	970	760	920	1080

Source: Long Island IMIS Feasibility Study

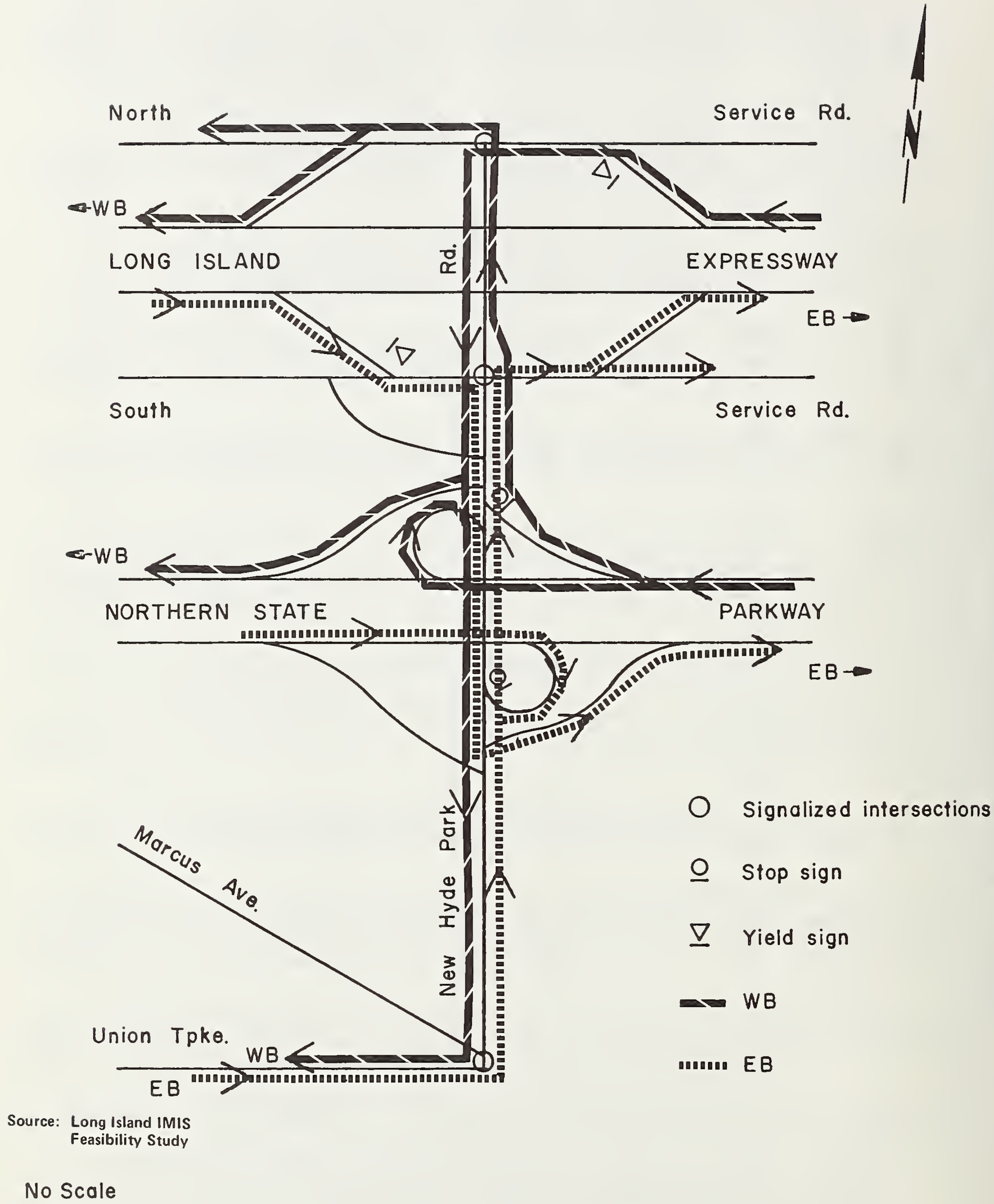

Figure 12. Example of Potential Diversion Paths

Table 24. Example of Available Capacity Calculation for Freeway Connections

FREEWAY CONNECTION EXCESS CAPACITIES

LOCATION	DIRECTION	AVAILABLE CAPACITY AT LEVEL OF SERVICE E 4-5 PM Ramp	4-5 PM Freeway	MIDDAY HOUR Ramp	MIDDAY HOUR Freeway
Long Island Expressway at Grand Central Pkwy.	From N to E	550	600	980	1840
	From W to S	470	660	720	3420
Long Island Expressway At Clearview Expressway	From N to E	1590	920	1650	1850
	From W to S	1470	2760	1560	3710
	From S to W	1300	1350	1450	2240
Long Island Expressway At Cross Island Parkway	From N to E	700	60	870	920
	From S to E	1070	200	1390	1020
	From W to S	690	630	810	2700
	From S to W	0	200	600	1400
Long Island Expressway At Northern State Parkway Connection	* From W to Pkwy	1050	1260	1500	3560
	From Pkwy to W	850	800	1090	1830
Long Island Expressway At Northern State Pkwy Connection Near Rt 135	Outbound to Pkwy	980	70	1210	2020
	Inbound to Pkwy	1000	1650	1140	2200
Long Island Expressway At Route 135 Exp.	From W to S	440	1530	1270	3390
	From W to N	1230	3950	1300	4280
	From E to N	1210	3760	1210	4090
	From N to W	1550	800	1650	1800
	From N to E	780	1230	1200	2450
	From S to E	1440	870	1550	2200
Long Island Expressway At Northern State Pkwy	From SW to E	1020	320	1310	1820
	From NE to W	1280	640	1510	1720
Long Island Expressway At Sagtikos State Pkwy	From N to W	1680	1800	1730	2570
	From W to N	1340	3190	1580	3320
	From E to N	1640	2030	1690	3110
	From N to E	960	70	1310	2820
Grand Central Parkway at Long Island Expsy.	See Long Island Expressway at Grand Central Parkway				
Grand Central Parkway at Clearview Expressway	From W&E to N	920	3550	1250	3800
	From N&S to W	1200	1300	1350	2170
	From W&E to N	920	3520	1250	3800
	From N&S to E	1050	170	1350	2440
Grand Central Parkway At Cross Island Pkwy	From W to N	1530	2490	1560	3150
	distrib N to W	1130	1460	1440	2330
	From E to N	530	1420	1150	2700
	distrib N to E	300	240	680	1930

* Thru traffic diversion capacity limited by Northern Parkway capacity east of Meadowbrook Parkway.

Source: Long Island IMIS Feasibility Study

corridor. The estimate is made for a typical peak hour. The results will be used subsequently to develop average trip length, assess diversion potential and estimate benefit reduction associated with fewer diversion points. In order to determine an origin-destination pattern for each facility a sequential listing (upstream to downstream) of all entrance and exit ramps is required along with a balanced hourly volume map. The output of the analysis provides the distribution of volume from each entrance ramp to each exit ramp. Appendix A gives the detailed procedure for generating the origin-destination pattern.

7.7 TRIP LENGTH ANALYSIS

The purpose of this analysis is to develop an estimate of the median trip length during a typical peak hour on the major limited access facility or facilities within the corridor. The results will be used subsequently to evaluate diversion potential and estimate benefit reduction associated with fewer diversion points. The inputs required to perform this analysis are an origin-destination worksheet and the distance of each ramp from the start of the corridor. An accuracy of 0.1 mile (0.16 km) is adequate. The output is a composite average of the median trip length for the corridor. Appendix B gives the detailed procedure for generating the trip length for any specified limited-access facility.

CHAPTER 8

ALTERNATE ROUTE ANALYSIS

8.1 INTRODUCTION

8.1.1 Objectives

- To analyze and rank the candidate list of alternate routes based on a set of quantitative criteria.

8.1.2 Inputs

- The baseline corridor map prepared after the initial route screening.
- The flow and incident characteristics cataloged by roadway and available from the corridor data base.
- The other categories of data available from the data base.

8.1.3 Outputs

- The candidate alternate routes to be considered as part of the IMIS network, ranked with respect to their capability to serve as effective alternates for their associated primary route(s).

8.2 PROCEDURE

Using the baseline corridor map, identify and list the candidate alternate routes associated with each of the major primary routes (limited access facilities) in the corridor. If an alternate route can logically serve more than one primary route, it should be considered separately for each case.

The ranking of the candidate roadways is performed in accordance with the quantitative factors shown in Table 25. The quantitative factors have been formulated so that higher numerical scores reflect more desirable routes. A set of these tables should be prepared for each route under consideration.

The total numerical score serves as a relative measure of comparison of the alternate routes with respect to their associated primary route(s). As will be discussed in a subsequent section, the alternative systems will be structured in part by using these ratings.

Scale factors as shown in Table 25 convert the raw value of each characteristic into a normalized value. These normalized values are then weighted and summed to obtain the overall score.

A set of nominal weighting factors has been assigned to each characteristic as shown in Table 25. These will serve as a priori estimates of the relative importance of each characteristic. If the traffic engineer desires to modify the weights to reflect conditions in his corridor, he is free to do so.

The nominal weights have been selected based on the experience of previous IMIS type design studies. It appears from this experience that available capacity and relative travel time are the most important factors in determining viability of an alternate route and these have each been given a weight of 0.2. Another significant factor is the mileage penalty incurred in taking an alternate and this has been given a weight of 0.15. Additional connectors between the alternate route and the primary route are considered to be an asset, since they add to the diversion flexibility. (This factor has been assigned a weight of 0.10). However, the connectors are only of value when they have some available capacity. To compensate for this, connectors are only included in the computation if they have a reasonable amount of available capacity. (A nominal available capacity value of 50 percent or more of the alternate route available capacity has been assumed as the dividing point.) Item 7 in the table provides the traffic engineer with the opportunity to judge other factors which, though difficult to quantify, are nonetheless important indicators of route quality. Accident frequency is included as a quality measure in this item. The composite weight of these additional factors is taken as 0.20.

The raw values to be inserted in the tables are obtained from detailed maps, land use maps, and traffic engineering analysis and the traffic engineer's experience. In particular, the traffic engineering anaysis should provide raw values for characteristics 2, 4, and part of 5 (connector capacities) in Table 25. In the event that data are not available for all the entries, the traffic engineer may choose either to estimate the value or to eliminate that factor and adjust the weight of the other factors appropriately.

The ranking of the alternate routes as specified by the overall score. obtained from Table 25 is the primary output of this task. The final overall ratings should be summarized into an "ALTERNATE ROUTE RANKING" Table, showing each primary route and associated ranked alternate routes. The summary will be used later in the development of the alternative system designs.

An example illustrating the calculations for ranking alternative roadways is provided using the network given in Figure 13. The overall length of the corridor extends from point L to M with a single limited-access freeway (the primary route) and two arterials available as possible alternates.

It should be recognized that this network is relatively simple in order to facilitate the ranking calculations presented; actual traffic corridor networks should be expected to be of greater complexity.

Table 26 gives the rating calculations for the alternate route consisting of roadway segments (I, G), (G, H), (H, E) and E, F). This alternate route is compared to the primary freeway route (I, F). All calculations are shown in Table 26. Tables 27 through 31 provide the flow and accident data (assumed). Note that these data are catalogued by route segment with respect to the individual corridor alternate and primary routes. Tables with these formats would be required for the other route segments of the corridor if additional rankings are to be considered.

Table 25. Worksheet To Compute Alternate Route Score

ALTERNATE CHARACTERISTICS	WEIGHT	RAW VALUE	SCALE VALUE	SCALE VALUE X WEIGHT
		(Portion To Be Filled Out In Scoring Process)		
1. Length of primary route bypassed in percent of primary route length	.05			
2. Average peak hour available capacity of alternate* in percent of primary route capacity	.20			
3. Length of primary route bypassed as percent of length of alternate (include length of connector routes)	.15			
4. Peak hour primary route travel time as percent of alternate route travel time (include connector route travel time)	.20			
5. Additional connectors. Average number of additional connectors per mile of primary route** (round off the nearest tenth)	.10			
6. Land use. Percent of alternate route dedicated to non-residential use.	.10			
7. Other factors. Grade from 0-100. Deduct for school zones, railroad grade crossings, high accident frequency and other qualitative factors.	.20			

CHARACTERISTIC	RAW VALUE	SCALE VALUE
#2	≥10	100
	9	90
	8	80
	7	70
	6	60
	5	50
	4	40
	3	30
	2	20
	1	10
	0	0
#5	≥1.0	100
	0.9	90
	0.8	80
	0.7	70
	0.6	60
	0.5	50
	0.4	40
	0.3	30
	0.2	20
	0.1	10
	0	0
#1,3,4,6,7	95-100	100
	90-94	90
	80-89	80
	70-79	70
	60-69	60
	50-59	50
	40-49	40
	30-39	30
	20-29	20
	10-19	10
	0-10	0

*Use the average (rather than the minimum) of the individual intersection excess capacities, to compensate for the added capacities that computerized signal control operations will provide.

**If the average available capacity of any connector is less than 50 percent of that of the alternate route, the connector should not be included.

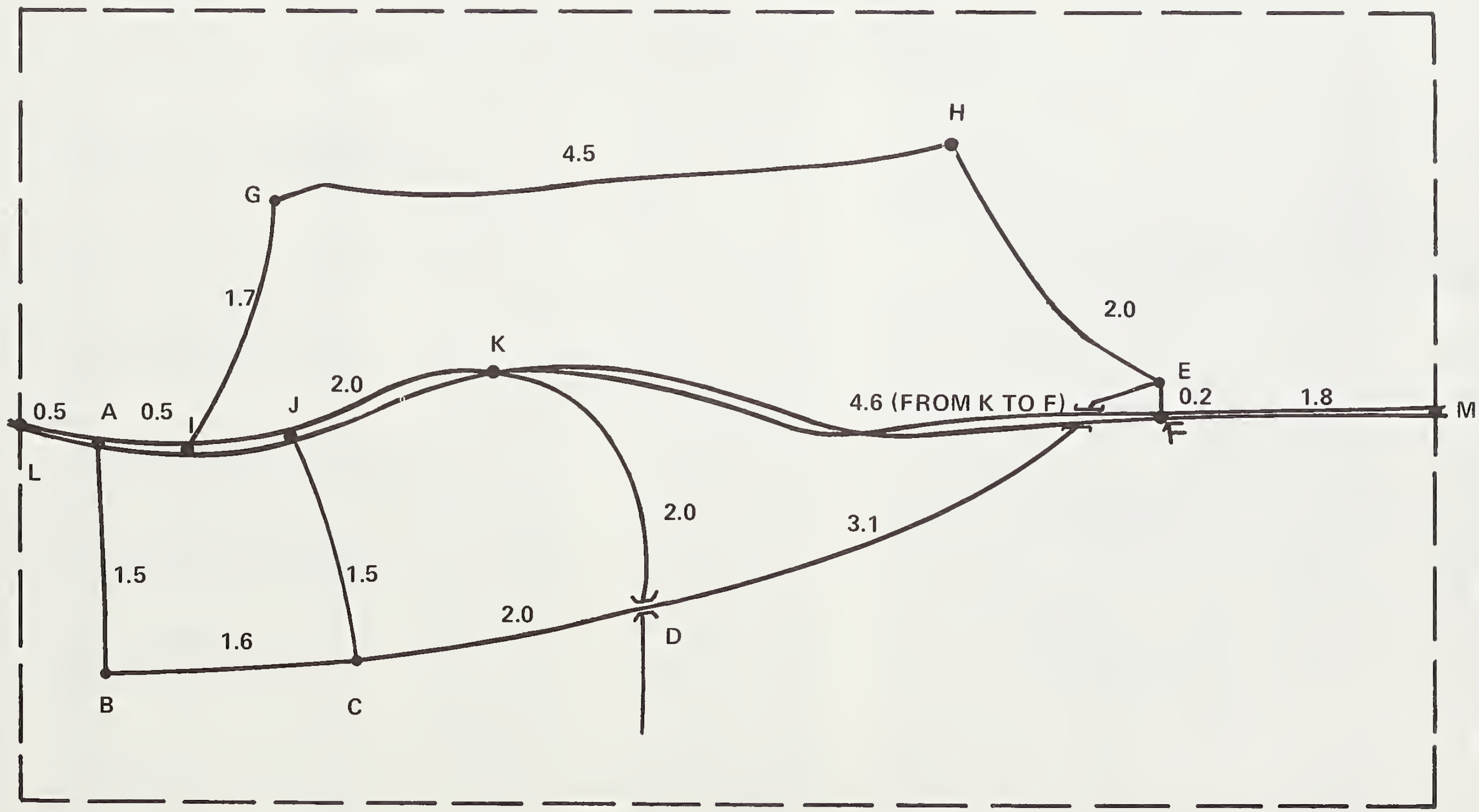

Figure 13. Roadway Network for Alternate Route Ranking Example

Table 26. Worksheet for Sample Route Ranking Problem

ALTERNATE ROUTE IGHEF COMPARED TO PRIMARY ROUTE IF					
Characteristic*	Calculation	Raw Value	Scale Value	Weight	Weighted Scale Value
1	a. length of primary route 0.5+0.5+2.0+4.6+1.8=9.4				
	b. length of primary route bypassed 2.0+4.6=6.6				
	c. % bypassed = $\frac{6.6}{9.4}$x100=70	70	70	.05	3.5
2	a. From Tables 27, 28 Average of (400,850,600,450,1100, 550,450,900) = 663				
	b. Primary route capacity =5100				
	c. 663/5100x100=8	13	100	0.20	20
3	a. length of primary route by-passed =6.6				
	b. length of alternate 1.7+4.5+2.0+0.2=8.4				
	c. (6.6/8.4)x100=79	79	70	0.15	10.5
4	a. From Table 29, primary route travel time = 2.7+6.9=9.6 min				
	b. From Table 29 Alternate route travel time = 5.1+6.7+4.8+0.8=17.4 min				
	c. 9.6/17.4x100=55	55	50	0.20	10
5	no additional connectors	0	0	0.10	0
6	From Table 30 Weighted Land Use % = $\frac{[(1.7x50+4.5x85+2.0x40+0.2x90)]}{(1.7+4.5+2.0+0.2)}$	67.3	60	0.10	6.0
7	Route passes 3 elementary schools and some quiet residential areas. Also 4 high accident locations (Table 31). Total grade about 20 out of 100.	20	20	20	4
		Total Score			54

*See Table 25 for definition

Table 27. Available Peak Hour Capacity* (Assumed for Example)

Route Segment	Available Capacity (V/hr)*
IG	400
GH	850
HE	600
EF	450

Table 28. Available Peak Hour Capacity, Turning Movements* (Assumed for Example)

Turn		Available Capacity (V/hr)*
From	To	
AI	IG	1100
IG	GH	550
GH	HE	450
HE	EF	900

*Level Of Service E

Table 29. Peak Hour Travel Times (Assumed for Example)

Route Segment	Travel Time (Min)
IG	5.1
GH	6.7
HE	4.8
EF	0.8
IK	2.7
KF	6.9

Table 30. Land Use (Assumed for Example)

Segment Miles		Non-Restricted (Percent)
IG	1.7	50
GH	4.5	85
HE	2.0	40
EF	0.2	90

(Note: 1 mile = 1.609 km)

Table 31. High Accident Locations (Assumed for Example)

Route Segment	# Of High ACC. LOC.
IG	1
GH	2
HE	1
EF	0

8.3 INTERPRETATION OF RESULTS

In the sample calculation, the total score obtained for the one alternate route considered was 54. While no calculation was performed for the second alternate route shown, suppose that its total score was 48. Then, the obvious implication of the ranking results would be that if one of the alternate routes is to be eliminated (e.g. for a lower cost alternative design), it should be the lower ranked route.

It should be kept in mind that alternate route scores should only be compared when associated with the same primary route. For example, suppose the corridor contained another freeway with two other alternate routes, and their scores were 46 and 40. This would not imply that the latter two routes should necessarily be excluded before considering exclusion of one of the other freeway's alternates.

Consider further the case where one alternate could serve two primary routes as, for example, when the alternate lies between the two primary routes. From the above set of 4 scores, suppose the common alternate's score was 54 with respect to one primary route, and 40 with respect to the other primary route. If one route were to be excluded for an alternative lower cost design, this should not be the one with the lowest score (40) because of its utility for the other primary route. Instead the route scored 46 (or perhaps the one scored 48) would be the more appropriate choice for elimination.

In summary, while a procedure for quantitative scoring of alternate routes is provided, the resulting numerical values cannot be mechanically applied to exclude routes. Rather, due consideration must be given to the specific circumstances (such as those noted above) if meaningful results are to be obtained. Thus, the final ranking of routes should be in the form of retention priority rather than a simple ordering of numerical scores.

CHAPTER 9

SELECTION OF ROADWAY NETWORK

9.1 INTRODUCTION

9.1.1 Objectives

- To assemble the corridor roadways into a set of identifiable IMIS networks suitable for implementation of the IMIS concepts.

9.1.2 Inputs

- General knowledge of corridor traffic operations.
- The set of candidate corridor roadways with their associated ranking for inclusion in the networks (from Chapter 8).

9.1.3 Outputs

- Maps showing each candidate network configuration. The maps should clearly identify major corridor routes (both limited access and arterial), and the principal connector routes.

9.2 GUIDELINES FOR NETWORK SELECTION

The assembly of the corridor roadways into identifiable networks is required for the later development of the IMIS candidate designs. It is generally recognized that the subsystem design and actual implementation of the IMIS functions will have a strong influence on the level of benefits and costs. This is not always recognized, however, with respect to the design of the roadway network. The inclusion or exclusion of certain roadways can be a major influence certainly on cost (since the roadway would require instrumentation) and also on performance since an included roadway would add the flexibility to utilize certain system functions (such as diversion and ramp metering) in a more effective manner. For example, if a high quality arterial which runs the total length of the corridor is excluded from all (or included in all) networks, there is no way to assess the importance of that roadway to the total transportation needs of the corridor. To make this assessment, two networks would have to be defined with the arterial included in one and excluded in the other. The inclusion of the arterial could provide an available alternate for traffic which is diverted (via ramp metering) from the primary limited access facilities. With the arterial excluded from the network, the flexibility of ramp metering as a control policy could be significantly reduced since metered traffic would have to wait at the ramp.

With these considerations in mind, the following guidelines can be used to define a set of network configurations:

- All limited access roadways which run the full length of the corridor should be included in all network configurations for corridors which are approximately 15 miles (24 kilometers) or less in length. For substantially longer corridors, one or more of the limited access facilities or segments thereof may be excluded, if appropriate, to produce a geometrically smaller corridor.

- The frontage roads (or service roads) associated with the primary freeways should be included in all network configurations, since they are in effect an integral part of the freeway's operation.

- Major arterials which run parallel to the primary freeways and extend over their entire length should be included in at least one of the candidate networks.

- Arterials which run parallel to the primary freeways for a short but significant portion of their length and which, if eliminated from all network configurations would limit the control flexibility of IMIS in that area of the corridor, should be included in at least one of the candidate networks.

Using the above guidelines, it is recommended that a minimal set of three network configurations be developed. The first and most extensive of these should generally include all of the candidate roadways (including connectors) remaining after the initial route screening process. The others should be defined as progressively smaller subsets of the first, with significant enough changes in each to provide a meaningful variation in cost. Judgment must be exercised in the process to insure that reasonable and coherent networks result. The ranking of alternate routes, as discussed in the previous section, provides guidance for treating these roadways on an individual basis.

As indicated previously, the network selection is an input to the development of the candidate IMIS designs, which will be addressed in Chapter 13. At this point it is noted that each network will be used to generate at least two designs, i.e., one containing all IMIS functions and maximum equipment complements and the other with minimum equipment and possibly some functions removed. Depending on corridor size and traffic characteristics, an intermediate candidate may also be considered.

CHAPTER 10

ESTABLISHMENT OF CONTROL AREA BOUNDARIES

10.1 INTRODUCTION

10.1.1 Objectives

- To partition the overall roadway network into subnetworks, each with a common control philosophy based on the control functions of ramp metering, diversion and traffic responsive arterial signal control. The purpose of the partitioning is to provide a basic structure for implementing the IMIS control functions with respect to an actual corridor with a specific roadway network.

- To develop the control probability factors for each subnetwork type. The basic purpose of "control probability" is to incorporate into the feasibility methodology a procedure for evaluating the real-time dynamic capabilities of IMIS. The need for this capability in an evaluative methodology directed at corridor surveillance and control systems is not generally recognized. The standard traffic analyses which assess system capability do not provide the needed indication of traffic variability that a real-time system can measure and respond to. The introduction of the control probability factor directly addresses system dynamic capabilities. Also calculated is a corresponding value of control volume shift capability.

10.1.2 Inputs

- The corridor network(s) with all freeways, arterials and connectors identified (from Chapter 9).

- The control probability model coefficients (from Chapter 7).

10.1.3 Outputs

- The corridor partitioned into a connected set of subnetworks. Each subnetwork encompasses a section of the corridor with a common set of IMIS control functions. The boundaries of the subnetwork define points at which a significant change in control function and philosophy occurs.

- A map of the corridor with the subnetworks defined.

- A tabulation of the control probability factors specified for each subnetwork type.

- A tabulation of the control volume shift capability.

10.2 PROCEDURE FOR ESTABLISHING CONTROL SUBNETWORKS

The extent to which IMIS control can be applied is a function of the roadway configuration. Since the roadway configuration usually varies in different portions of the corridor, it is necessary to partition the corridor into segments, according to the type of control that can be applied. Since these segments represent portions of the overall network, they are termed "subnetworks".

The set of IMIS control functions includes ramp metering, diversion, and responsive arterial signal control. Where the roadway configurations permit, the controls may be applied in combination; otherwise they must be applied singly.

The partitioning of an IMIS corridor can be accomplished with three generic subnetwork types, defined as follows:

- Type 1 - This consists of two (or more) freeways without any service roads or parallel arterials (i.e., no other alternate routes). As such, it can only provide the freeway-to-freeway diversion control function.

- Type 2 - This consists of a single freeway with a service road or a parallel arterial. Ramp metering can be used, along with responsive signal control on the service road or arterial. While it is possible to also have diversion from the freeway, it is assumed that ramp metering is the primary control, and that available capacity on the service road (or arterial) will be used by vehicles diverting from the metered entrance ramps.

- Type 3 - This consists of two (or more) freeways with at least one service road or parallel arterial. This type of subnetwork permits the full complement of IMIS controls to be applied. The case of one freeway with two or more arterials (one of which may be a service road) may also be considered as a type 3 subnetwork since the presence of the second arterial should provide diversion capability as well as ramp metering.

An example of corridor partitioning is provided with the aid of Figure 14. (This figure is a fold-out map. For convenience, it is located at the end of the report, page 245). In the figure, sections of the corridor are shown outlined and labelled in accordance with the type of subnetwork that they represent. In the labelling, the "A" and "B" notations are used for illustration purposes. For example, 2A contains a freeway with service road while 2B contains a freeway with arterial. Both, however, would be considered as Type 2 subnetworks for control capability purposes.

It should be noted that subnetworks 2A and 2B were defined primarily to illustrate the Type 2 subnetwork. Ordinarily, these two would be grouped together to form a Type 3 network, to provide full IMIS control capability. In general, partitioning of the corridor

is done only along the lengthwise dimension, to obtain this maximum capability. If roadways or diversion points are deleted in some alternative designs, the partitioning must be reviewed to determine whether subnetworks change type.

The three types of subnetworks will normally allow any given IMIS corridor to be partitioned without any significant network omissions. As a "rule of thumb", the overall size of the subnetworks should not be smaller than 4 to 5 miles (6.4 to 8.0 km) in length.

10.3 PROCEDURE FOR DETERMINING CONTROL PROBABILITY FACTORS

The control probability factors account for variability in traffic flow and represent, in essence, the fraction of time that IMIS control can be exercised. Since different subnetwork types permit different levels of control to be instituted, the control probability factors for each must be considered separately. In addition, control capability varies with prevailing roadway conditions (peak period normal flow, peak period incident, off-peak period incident), and these must be considered separately as well for each subnetwork type.

The procedures for establishing the control probability factors are provided in Appendix C. A typical or standard set of factors is given in Table 32. This set can be used if so desired as a substitute for developing a set unique to the user's corridor. The appendix should be first reviewed, however, to provide an understanding of the concept and a basis for this judgment.

Table 32. Typical Control Probability Factors

Subnetwork Type	Control Function	Roadway Operational Conditions		
		Peak Period Normal Congestion	Peak Period Incident	Off Peak Period Incident
Type 1 Two Or More Freeways	Diversion Only	.18	.3	1.0
Type 2 Single Freeway With Service Road Or Arterial	Ramp Metering, Signal Control	.3	1.0	1.0
Type 3 Two or More Freeways With At Least One Service Road Or Arterial	Diversion, Ramp Metering, Signal Control	.51	1.0	1.0

For example, the assumption is made that during peak period incident conditions, ramp metering will be exercised in any affected subnetwork possessing this capability, virtually 100 percent of the time (control probability factor = 1.0). The philosophy behind this assumption is that if a freeway incident occurs during peak periods, the ramp metering capability, coupled with appropriate computer control of signals on the alternate (service road or arterial) can and should be used in an attempt to alleviate a severe freeway problem. Inherently, this implies that a net benefit can be achieved with this control action under these conditions. It should be specifically noted, however, that the control probability factor does not inherently imply a magnitude of control (e.g. a metering rate), or a magnitude of benefit. Rather, it specifies the fraction of time that a given control policy is presumed capable of achieving benefits for the given conditions in the defined subnetwork type. It also accounts statistically for factors relating to traffic variability, such as the probability that one freeway flow is at some level higher than its average while the other freeway flow is correspondingly lower. Such factors are of critical importance during the peak demand periods.

10.4 PROCEDURE FOR DETERMING CONTROL VOLUME SHIFT CAPABILITY

In Chapter 7, the control probability model coefficients (mean flow and standard deviation) were calculated. Of particular interest at this point is the measure of traffic variability, i.e., the standard deviation, σ_Q. This parameter has been used in the development of the control probability factors by serving to define the boundaries of the three flow regions (regions A, B, and C) as described in Appendix C. It is now used to determine a mean value of control volume shift capability (ΔQ), that is, the number of vehicles per lane per hour that can be transferred from one freeway to another. (The percentage of time that the shift can be made is determined by the control probability factor).

For a roadway operating in flow Region A, the flow is substantially below the mean value and thus there is capacity available for control. The amount of the available capacity is the difference between the mean flow in Region A and the flow level at the start of Region C (or end of Region B). That is, it is assumed that the roadway can accept additional vehicles up to but not beyond the point that would cause it to enter Region C.

Since the mean flow level in Region A is one standard deviation* below the overall mean flow level (Q), and the B/C boundary line is 1/2 standard deviation above Q, the control volume shift capability is the sum of these distances, or 3/2 σ_Q. For example, if in Chapter 7 the average value of σ_Q was calculated to be 100 vehicles/lane/hour, the control volume shift capability, ΔQ, would be 150 vehicles/lane/hour. This value of ΔQ is applicable for shifting vehicles between freeways and is used in Chapter 15 to determine the overall control volume shift for a given network configuration.

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*The units are in vehicles/lane/hour

CHAPTER 11

REVIEW OF SYSTEM FUNCTION AND CONTROL POLICY

11.1 INTRODUCTION

11.1.1 Objectives

- To assess jurisdictional preferences regarding implementation of control functions.
- To determine jurisdictional constraints regarding selection of roadways for the corridor network.
- To determine requirements and constraints for interfacing with existing traffic surveillance and control systems.
- To verify that IMIS will support local transportation policy.

11.1.2 Inputs

- Local goals, objectives and transportation policies.
- Description of existing traffic systems.
- Comments from all involved agencies with regard to impact of IMIS on their jurisdictional operations, transportation and otherwise.

11.1.3 Outputs

- List of policy elements to be considered in structuring alternative systems.
- Plan for interfacing with existing surveillance and control systems.

11.2 DISCUSSION

This task serves as a point in the methodology where specific consideration and accommodation of local transportation policies and constraints can be incorporated into the IMIS designs. Inclusion of input from local agencies provides a sound basis for making the methodology responsive local policies.

The inclusion of local input can take many forms and can address widely different local concern. Each jurisdiction has distinct goals and objectives with

respect to its transportation services. These may reflect various geographic, socio-economic, legal and environmental factors or simply be the result of historical traffic and transportation practices.

Informal jurisdictional policies may have evolved from long-established practices such as incident/accident management regulations followed by law enforcement and towing concerns, or emphasis placed on public transportation, or emphasis placed on environmental issues. Where significant changes to arterial operations are expected (for example: in channelization, timing/phasing or parking policies), local agencies must be provided an opportunity to influence these plans.

In this task, the user should address the level of interaction to be maintained between the IMIS corridor-wide system and pre-existing or planned local signal system improvements. A review with local agencies would establish the necessary ground rules for the transfer of control signals to roadside. For example, local agencies may wish only to be kept informed of control actions being implemented on roadways within their area. Additionally they may require an intervention capability in order to maintain effective control at all time over 'their' roadways.

This task requires the user to review all stated and unstated policies of his jurisdiction. Those which appear to impact the design and implementation of IMIS must be noted. Finally, required modifications must be incorporated into the system design process.

CHAPTER 12

EQUIPMENT SELECTION FOR IMIS SUBSYSTEMS

12.1 INTRODUCTION

12.1.1 Objectives

- To select representative equipment types for use in the feasibility study, to the extent necessary to develop unit cost data.

- To develop corresponding unit cost data.

12.1.2 Inputs

- Baseline corridor map (from Chapter 5).

12.1.3 Outputs

- Equipment types and/or configurations for each subsystem.

- Unit cost data (capital, maintenance and operating)

12.2 OVERVIEW

The purpose of this chapter is to identify suitable equipment and equipment configurations for the system, so that cost data can be developed. Associated trade-off factors are discussed and, where applicable, approaches are recommended to minimize the effort necessary to achieve the desired outputs. Procedures or guidelines for estimating unit costs are presented and typical values are given which in most cases may be used as an alternative to acquiring costs from outside sources.

It should be noted that all typical cost data provided is representative for the 1977 time frame. Depending on when the feasibility study is performed, these values (if used) should be appropriately adjusted to the current year. Application of an average annual inflation rate should be adequate for this purpose.

The following subject areas are treated in this chapter:

- Variable Message Signs

- Fixed Signs

- Highway Advisory Radio
- Entrance Ramp Control
- Freeway Surveillance
- Arterial Surveillance and Control
- Other System Surveillance
- Motorist Aid Callboxes
- Pre-Trip/Enroute Informations Services
- Equipment Cabinets

The communications and control center areas are treated in Chapter 13 (Development of Alternative Preliminary System Designs), sinch their requirements are dependent on the overall system configuration.

12.3 VARIABLE MESSAGE SIGNS

12.3.1 Introduction

Variable message signs represent the primary source of real-time motorist information in IMIS. These signs, placed at key locations along the major roadways of the network, will indicate prevailing traffic and roadway conditions and, where applicable, provide alternate routing information for the motorist.

For the most part, placement of variable message signs in the corridor will be dictated by the location of diversion points. Maximum cost-effectiveness is achieved by placing signs upstream of the diversion points, since the signs can then serve to provide general traffic advisories as well as diversion information. Other signing locations can, of course, be added if needed to address special problems or problem areas located outside of the range of influence of the "diversion" signs.

To be effective, the variable message signs must be readable far enough upstream to give an approaching motorist adequate time to read and understand the message before it passes out of his field of view. The time required will vary with the length and complexity of the message. Assuming that a minimum of 10 seconds is required for an average motorist to read a 3-line message, then at freeway approach speeds of 55 miles per hour (88 km per hour), the sign must be readable at a distance of about 800 feet (244 meters). An approximate "rule of thumb" to determine required letter height is 1 inch per 50 feet (2.5 cm per 15 meters) of distance. Thus, required letter height is about 16 inches (41 cm). Standard letter heights are usually 15 and 18 inches (38 and 46 centimeters); the

larger value is preferred to increase reading distance. On arterials, approach speeds are slower and letter heights of 10 to 12 inches (25 to 30 centimeters) are usually adequate.

There is a large variety of variable message sign types; however, not all are suitable for the applications intended in an IMIS corridor. The following paragraphs briefly discuss the major types, their capabilities and limitations, and the important trade-off factors. An example of a typical cost comparison for the two most promising types is included. Next, signing configurations are discussed along with an example of a generic configuration which should satisfy most of the diversion point requirements. Finally, a procedure for establishing the signing configurations and determining unit costs is given. Again, an example is provided of a cost estimate for a typical signing installation, including capital costs of the sign and structure, and associated maintenance and operating costs.

12.3.2 Sign Types and Trade-off Considerations*

The different types of variable message signs which are suitable for outdoor use may be grouped according to external appearance into the following three categories:

- Roller-shade (scroll)
- Rotating drum
- Matrix

Roller-shade signs make use of a continuous belt upon which a series of messages is printed. The belt is rolled up between a storage drum and a take-up drum. When a message is selected, the belt is driven to the proper position so that the message appears in the display window. This type of sign provides maximum flexibility in message formatting. There are no restrictions on the characters, symbols, geometric shapes, and colors that can be used. Only a limited number of messages can be used on a belt (typically a maximum of 12). The sign is internally lighted and power consumption is relatively low. A major disadvantage of the roller-shade sign is its physical size. Because of the drums and the control mechanism required, the ratio of display area to the frontal area is small. In some present designs, a frontal area of 6 feet by 7 feet (1.8 meters by 2.1 meters) is required to provide a display area of 4 feet by 4 feet (1.2 meters by 1.2 meters). No signs of this type are presently made large enough to accommodate the requirements for freeway diversion applications (e.g. two or three lines with 15 to 18 inch (38 to 46 centimeters) character height). The signs can be used in combination; however, the resulting overall size and complexity can make this type of sign impractical for such applications.

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*A good reference document on the subject is "Variable Message Signing for Traffic Surveillance and Control, A State of the Art Report," by Warren Dorsey, Report No. FHWA-RD-77-98, January 1977.

Rotating drum signs are made up of one or more multi-faced rotors (drums) upon which message panels are attached. The drums are pivoted at both ends, providing a longitudinal axis of rotation for changing the drum face (message) exposed. Most present drums have three or four faces, although a hexagonal (six-faced) drum is available. Multi-line signs are produced by stacking a series of drums in a single enclosure. In this case, the drums may each be driven by individual motors, or by a single motor with a chain drive arrangement. Enclosures for rotating drum signs are not environmentally sealed, probably because part of the drum must extend beyond the sign face during the rotation cycle. As such, it is susceptible to collecting dust and dirt on the sign face as well as within the enclosure. To prevent ice build-up and freezing in cold climates, heating coils must be used around the periphery of the drum opening and where necessary, additional coils are embedded in the lower portion of the enclosure. Rotating drum signs have a similar degree of flexibility in message formatting as the roller-shade sign. The major limitations are slow message changing speed (as much as 30 seconds), limited number of messages, potential environmental problems, and mechanical complexity. Rotating drum signs are normally illuminated externally for nighttime operation.

There are several types of variable message signs available in the matrix category, i.e., bulb, disc, flap and fiber optics. The type most widely used thus far in traffic applications is the (incandescent) bulb matrix. It is manufactured in sizes large enough for use on high-speed highways and has good readability under a wide range of ambient light conditions. The sign display generally consists of a number of modules (one module per character) placed side-by-side in a metal housing. Each module is typically a matrix of 35 bulbs (5 columns of 7 bulbs each), with appropriate bulbs illuminated to form any alphanumeric character. Horizontal louvered sunscreens are sometimes used to improve readability and prevent phantom images caused by sunlight shining directly into the sign face. Generally, the sign enclosures are sealed and waterproof, also some use a water drainage system. The bulb matrix sign can be obtained with an unlimited message capability.

A potential problem with the bulb matrix sign is bulb maintenance. Although the bulbs are easily replaced from the front of the sign, periodic servicing is necessary because of the large number of bulbs used and the fact that the bulbs will be illuminated a large portion of the time. (A 3-line, 20-character per line sign will contain 2,100 individual bulbs.) In addition, relamping of the entire sign is usually done every one to two years for preventive maintenance. Another disadvantage of the bulb matrix sign is its relatively high power consumption. Typically, the sign uses 25 watt lamps. For the 3-line sign noted above, if 30% of the bulbs were used for an average message, the sign would use 0.3 x 2100 x 25, or 15.75 kilowatts. This factor is important both from an operating cost and energy conservation point of view.

The reflecting disc matrix sign produces messages in the same manner as the bulb matrix sign, except that it uses discs in place of bulbs. The discs have a basic background color on one side and are coated with a reflective material on the other. The discs are pivoted and flipped from one side to the

other, usually by electromagnetic means. Disc matrix signs are now being used in outdoor traffic applications. Their high reliability was proven earlier in outdoor advertising usage, where messages are changed up to four times a minute, continuously throughout the day.

Disc signs must be externally illuminated for nighttime hours; however, only a modest amount of power (about 1000 watts) is required for this purpose. A major advantage is that once a disc is flipped, it remains stable in that position - thus, virtually no power is needed to sustain a given message. Power to change messages is also trivial. A sealed enclosure provides environmental protection.

Present disc matrix signs "write" message on a character-by-character, line-by-line basis. This type of writing could be considered somewhat of a disadvantage relative to the bulb matrix instantaneous message change capability, particularly for a "flashing message" display. However, writing speeds for the disc sign are quite fast, normally less than one second per line, so that this should not constitute a significant problem.

A relatively new type of matrix sign uses electrostatically positioned vanes to form a desired message. The vanes are moved into either of two positions: erased (hidden from view) or written (in view). The sign face is made up of modules, each containing a specified number of vanes (typically 100). The modules fit closely together horizontally and vertically, and thus characters may cross module boundaries and be any size or shape. From close range, the sign face has a "mosaic tile" appearance. The display itself is passive; that is, it does not emit light but rather controls the passage and reflection of light. Power consumption for vane operation is low, but the sign requires illumination (usually internal) for low ambient light conditions. Sign maintenance requirements should be relatively small. The sign enclosure is sealed for environment protection.

One disadvantage of the vane matrix sign is that it has a somewhat limited viewing angle since the vanes extend outward (edge toward the viewer) in the "erased" position. Also, the sign operates from a high voltage source (typically thousands of volts) and, therefore, requires a high voltage enclosure to protect the electrostatic vane mechanism from the effects of moisture and dust. Since electrostatic parts tend to attract dust, the enclosure must be carefully sealed.

Perhaps the major disadvantage of the vane matrix sign is associated with its message writing speed. Messages are written sequentially across the sign face from left to right. A present standard display containing 120 columns of vanes requires 30 seconds for the entire display to change. While the next generation of signs may be improved in this regard, at present this factor represents a serious drawback for an IMIS application.

Another relatively new type of matrix sign uses fiber optic light pipes to guide light from a single lamp to the matrix points that make up the selected legend. Other legends are produced by switching to another lamp. The major advantages claimed for this type of sign are lower maintenance and power consumption (compared to the full bulb matrix sign) because fewer bulbs are used,

and reduced phantom images and more readable characters because of the narrow light beam. A disadvantage is that the number of messages is limited and it is difficult to change or add new messages. The fiber optics matrix sign technology is in a fairly early stage of operational use and believed not yet to have been applied to large freeway-type signs.

The trade-off factors considered to be of major importance in selecting the variable message sign type are:

- Ability to accommodate message complement requirements and flexibility to change messages or incorporate new messages without hardware modification;

- Proven operational capability;

- Cost, including capital, maintenance, and operating.

For an IMIS application, the variable message signs must provide an extensive message complement so that a capability exists to provide appropriate information to the motorist for the wide spectrum of conditions which can occur. In fact, in most instances, an "infinite" message capability is desired, that is, the ability to display virtually any message, or equivalently, to have the capability to update or change a set of pre-stored messages without replacing components.

The rotating drum and roller-shade categories can be eliminated as candidate types due to their inability to meet the message complement and flexibility requirement. In the matrix category, the fiber optics type has a similar shortcoming, coupled with its lack of proven operational capability and probable non-availability as "off-the-shelf" equipment; thus, this type can be eliminated.

Of the remaining types (bulb, disc and vane matrix), the vane matrix appears least desirable because of the operational factors noted earlier, particularly the message writing speed. Thus, the bulb and disc represent the primary candidates, with the major trade-off issue being cost.

Present experience is that the bulb matrix is somewhat less expensive to buy, but more expensive to operate and maintain. If it is desired to perform a cost trade-off study, capital and maintenance cost data may be obtained from sign manufacturers. (Operating costs can usually be calculated.) The study can be in the form of a unit basis for each sign configuration, i.e., 1-line, 2-line, 3-line signs with specified number of characters per line. (Common elements such as sign structure and installation need not be included at this point, although they must be considered later when system unit signing cost are developed.)

A sample of such a unit cost trade-off study is shown in Table 33. For the most part, the table should be self-explanatory. Two usage levels for the bulb sign (normal and low) were used to account for the fact that message lengths (and thus number of bulbs illuminated) would vary, particularly between daytime and nighttime, with shorter messages normally the case for the latter. A single

Table 33. Example of a Cost Trade-off Analysis for Lamp and Disc Matrix Signs

A. CAPITAL COSTS[1]

SIGN CONFIGURATION	UNIT SIGN COST LAMP	UNIT SIGN COST DISC
1 Line Insert, 10 char/line[2]	$ 8,000	$ 9,000
2 Lines, 20 char/line[3]	26,000	33,000
3 Lines, 20 char/line[3]	37,000	48,000
4 Lines, 20 char/line[3]	43,000	62,000

B. OPERATION COSTS (PER YEAR) ASSUMPTIONS

- variable message sign operation - 24 hours
- normal usage - 12 hours
- low usage - 12 hours
- cost of power = 0.6 per KWH

SIGN CONFIGURATION	UNIT OPERATIONAL COST PER YEAR LAMP[4]	UNIT OPERATIONAL COST PER YEAR DISC[5]
1 Line	$ 838*	$ 219**
2 Lines	1,796	307
3 Lines	2,628	438
4 Lines	3,548	526

*Sample calculation - Lamp Matrix Sign ($0.07/hr x 12 hrs - $0.03/hr x 12 hrs) 365 days = $438, plus $400 for external illumination (12 hrs/day) for fixed legend cases

**Sample calculation - Disc Matrix Sign ($0.05/hr x 12 hrs (nightime)) 365 days = $219

C. MAINTENANCE COSTS (PER YEAR)[6]

SIGN CONFIGURATION	UNIT MAINTENANCE COST PER YEAR LAMP	UNIT MAINTENANCE COST PER YEAR DISC
1 Line	$ 600	$ 270
2 Lines	1,200	990
3 Lines	1,800	1,440
4 Lines	2,400	1,860

Table 33. Sign Cost Trade-Off Analysis (Continued)

D. COST SUMMARY

SIGN COSTS	1-LINE		2-LINE		3-LINE		4-LINE	
	LAMP	DISC	LAMP	DISC	LAMP	DISC	LAMP	DISC
A. Capital	$ 8,000	$ 9,000	$26,000	$33,000	$37,000	$48,000	$43,000	$62,000
B. Operational	838	219	1,796	307	2,628	438	3,548	526
C. Maintenance	600	270	1,200	990	1,800	1,440	2,400	1,860
D. Cap. Recov. of (A)*	1,052	1,183	3,418	4,339	4,865	6,311	5,653	8,151
E. Total Yearly	2,490	1,672	6,414	5,636	9,293	8,189	11,601	10,537
F. Current Value of (B+C)	10,938	2,273	22,788	9,865	33,680	14,284	45,241	18,148
G. Total Current Value (A+F)*	18,983	11,273	48,788	42,865	70,680	62,284	88,241	80,148

(*Based on 10% interest rate, 15 year life)

NOTES

(1) Capital costs for both sign types do not include sign control equipment located at the system control center facility, mounting structure or installation costs.

(2) Capital costs for both sign types based on providing a minimum of four (4) messages.

(3) Capital costs for both sign types based on signs having the capability of storing at least 8 messages per line with the feature of being able to place new messages into storage from the IMIS control center.

(4) Normal usage cost based on 25% of lamps illuminated for message. Low usage cost based on 10% of lamps illuminated for message. Each usage applies for 12 hours.

(5) Operating cost is for nighttime operation. Disc sign does not draw power in a static state. Power consumed to change message is neglegible.

(6) Values based on discussions with sign manufacturers. For the disc sign, value is 3% of the capital cost of the sign (excluding mounting structure and installation).

average case could probably be used as an alternative. The 1-line sign (10 characters) is used as an insert in a fixed guide sign panel, as will be subsequently discussed under signing configurations.

The sample shown was taken from the IMIS study for the Long Island, NY corridor. For this case it can be seen that, on a total equivalent cost basis, the disc sign has the overall cost advantage. Also, from the using agency's viewpoint, its operating and maintenance costs (non-eligible for federal participation) are substantially lower. Energy conservation aspects also favor the disc matrix type of sign.

It is not necessary to make a final sign type selection during the feasibility study, but rather to estimate typical costs for signing. If a preference does exist for a given type, that type may be used for estimating purposes. Alternatively, an average cost may be used, or the higher cost used for more conservatism. In any case, it is not expected that the difference would affect the final outcome of the feasibility study.

12.3.3 Variable Message Signing Configurations

Variable message signing configurations can vary from location to location within an IMIS corridor, depending upon the functional requirements for the sign and the roadway geometry. For example, a single relatively small sign may suffice on an arterial, whereas normally at least two signing stations are required on a freeway for a given diversion point (similar to standard guide signing practice). Furthermore, more complex diversion points may require larger signs than the more straightforward cases. Therefore, some estimate of the different types of sign configurations expected to be encountered in the corridor should be made so that representative unit costs for each can be obtained. (This information will also be used later to determine total system signing costs, by multiplying each category by the average unit cost and summing the results.)

One generic configuration which should satisfy most of the freeway diversion point requirements consists of two signing stations, one located 1/2 to 1 mile (0.8 to 1.6 kilometers) upstream of the diversion point, and the other at the approach to the diversion point. The first signing station contains a suitably mounted multi-line variable message sign, which provides traffic conditions and alternate route information (when appropriate). The second station uses one or more single-line variable message inserts, incorporated into a fixed guide sign(s), to serve as confirmation for the upstream sign. Two typical examples of this type of configuration are shown in Figures 15 and 16. Both indicate the addition of a new 3-line variable message sign. In Figure 15, the previously existing roadside guide sign approaching the diversion point is shown modified to incorporate the single line variable message insert for confirmation. In Figure 16, the existing guide signing approaching the diversion point is a series of panels mounted on a sign bridge. Here, as shown modified, single-line inserts have been added to two of the sign panels. In this case (which was proposed for the Baltimore "Single

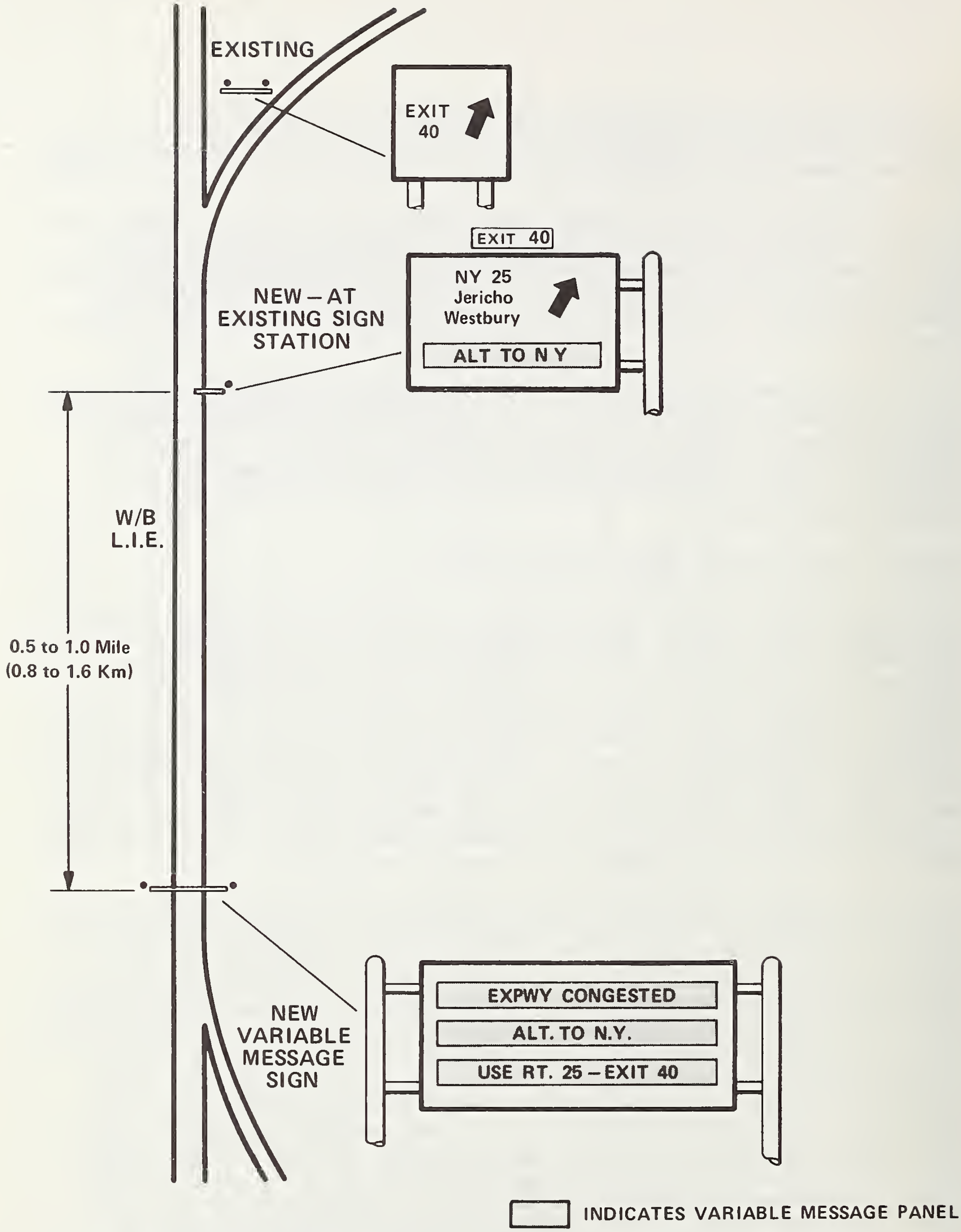

Figure 15. Typical Signing Configuration

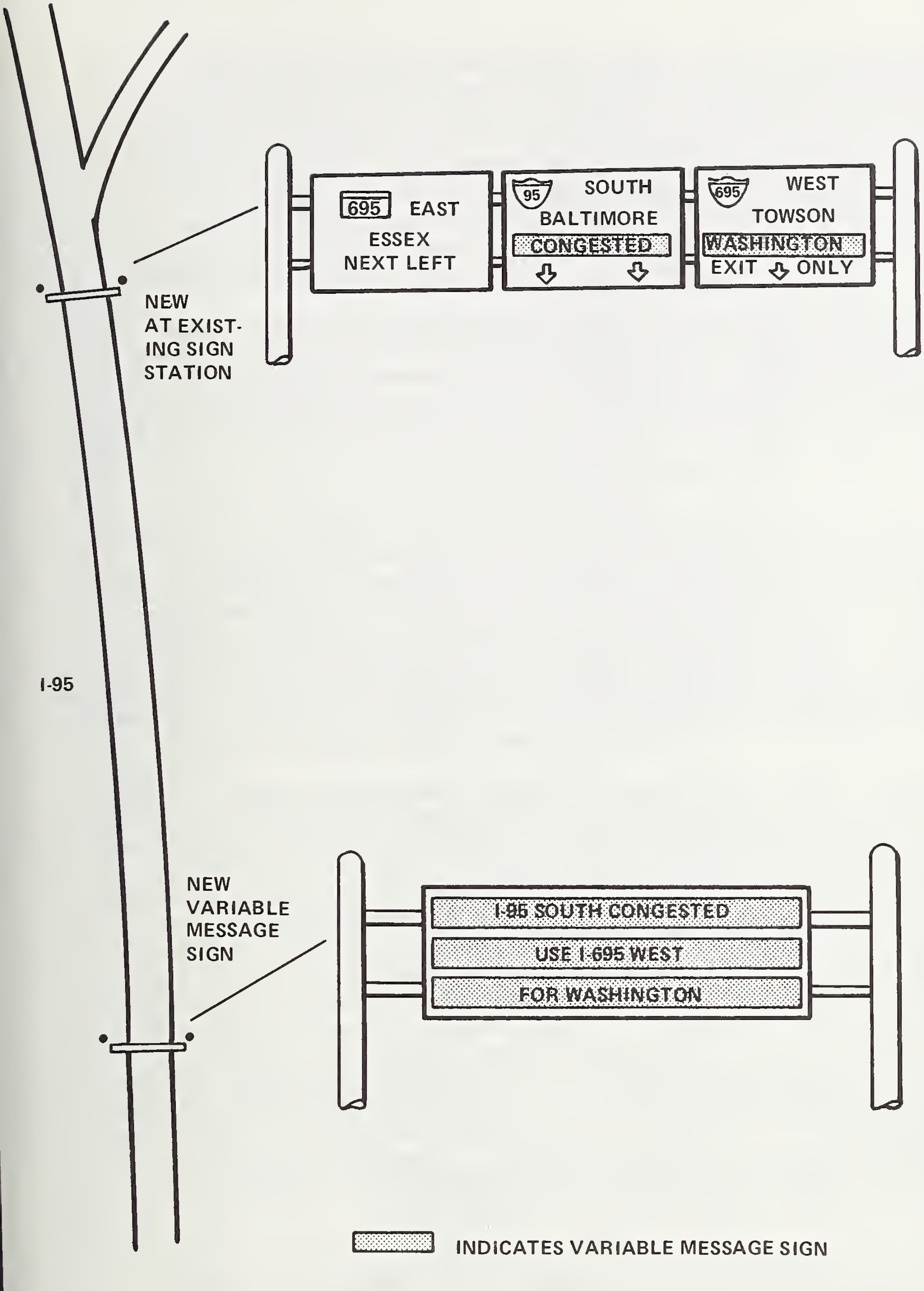

Figure 16. Typical Signing Configuration

Point" Diversion System*), the original center panel included "Washington" as part of the fixed legend. This was deleted in the revised panel to avoid possible confusion to the motorist. Under non-diversion conditions, the center variable message insert would read "Washington" (since I-95 is the normal route) and the right-hand insert would be blanked. In general, the new fixed guide signs are replacements for the existing signs, containing the original fixed legend, as appropriate, but increased in size as necessary to accommodate the variable message inserts. Mounting structures will probably require replacement as well, unless a sufficient safety factor exists to accommodate the increased wind loading and dead weight.

The most common size expected for the full matrix sign is 3 lines, with about 20 characters per line. This should provide sufficient flexibility in most cases to present the traffic condition and diversion recommendation in language consistent with the existing fixed signing. For the variable message insert, it is expected that a single line of 10 to 12 characters will suffice for the confirmation.

12.3.4 Procedure for Determining Signing Configurations and Unit Costs

The approach recommended for determining the different sets of typical signing configurations is as follows: using a map of the corridor, inidicate (.e.g, by colored pencil) all of the routes considered as candidates for potential inclusion in the system. Mark the points which can be considered as possible diversion points and note the width of the roadway or number of lanes approaching the diversion point. (The latter information is used for determining typical mounting requirements, e.g., if sign bridge, approximate span required.) A set of signing plans will be useful to indicate the existing fixed guide signing for cases where inserts are to be added. Alternatively, this information can be obtained from a field trip.

Number each diversion point (or signing location) on the map. Prepare a worksheet to record signing requirements. (The worksheet can be of a form similar to that shown in Table 34.) Next, consider each location from a functional viewpoint and estimate the number of lines required for the multi-lane variable message sign and the number of inserts (if any) for the downstream station. It is expected that the typical configuration noted earlier will be applicable to most freeway locations. The present objective is to determine how many other categories should be considered for unit cost estimating. Some judgement must be used to avoid an unduly large number of categories, since small refinements are not required in the feasibility study. For example, if most multi-line signs require a sign bridge to span 3 lanes, but one or two require a 4-lane span and another only requires a 2-lane span, these differences can be neglected and only the one category (3-lane span) applied to all. Similarly, if 2 single-line inserts are used at most locations, but there is an occasional application for one or three, the predominant case may be used as an average. In general, one can assess whether the use of such averaging will have a significant effect on cost. Full advantage of simplicity should be taken when appropriate.

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*Sperry Systems Management, "Final Design Report, Diversion of Intercity Traffic at a Single Point", November 1973 (Revised September 1974).

Table 34. Typical Worksheet for Estimating Signing Configurations

LOCATION I.D. NUMBER	MULTI-LINE VARIABLE MESSAGE SIGN						VARIABLE MESSAGE INSERTS			
	NO. OF LINES REQUIRED			TYPE OF SUPPORT REQ'D			NUMBER REQ'D	EXISTING SUPPORT TYPE*		
	2-LINE	3-LINE	4-LINE	BRIDGE	CANTI-LEVER	ROAD-SIDE		BRIDGE	CANTI-LEVER	ROAD-SIDE
1		✓		✓			2	✓		
2		✓		✓			1			✓
3	✓				✓		0			
.										
.										
.										
(ETC)										

MULTI-LINE SIGNS ARE 20 CHARACTERS/LINE, EXCEPT WHERE NOTED OTHERWISE
SINGLE-LINE INSERTS ARE 10 CHARACTERS, EXCEPT WHERE NOTED OTHERWISE

*GENERALLY, TO BE REPLACED BY A NEW STRUCTURE OF THE SAME TYPE

Once the number of configurations has been established, unit costs should be developed for each, including capital, maintenance and operation. A sample calculation is shown in Table 35. For the variable message sign cost, only the cost of the sign, including its housing, should be used. (The cost of the controller is included with the computer costs at the central facility.) The best sign cost estimates are obtained from manufacturers. However, the values shown previously in Table 33 may be used if such information is not readily obtainable.

The results of the calculations should be summarized in a table, indicating the different configurations and the capital, maintenance and operating cost for each.

12.4 FIXED SIGNS

12.4.1 Introduction

Many freeway motorists, especially those who are less familiar with a given area, are reluctant to leave their pre-planned route even when they encounter severe congestion. This reluctance, at least in some cases, stems from a feeling that once they stray from the "known" into the "unknown", there is probability that they will get lost.

Diversion from freeways (when necessary) is a primary IMIS control function. It's success will ultimately depend on acceptance of its overall credibility by the motorist. It therefore behooves the system designer to insure that all motorist information needs are satisfied. Thus, when a motorist complies with a diversion message, he should be provided with adequate route guidance/route confirmation signing. For example, suppose that a motorist is advised (via variable message sign) to use an alternate route (say, "Broadway Blvd", Exit 19). If the motorist after leaving the exit ramp, comes to a signalized intersection, he should not have to wonder which way to go. A sign should be installed (if not already there) containing an appropriate legend (e.g. "to Broadway Blvd"). Such signing should similarly be provided at any other "decision point" along the way. Furthermore, once on the alternate route, additional route guidance signing should be provided periodically to confirm to the motorist that he is still on the proper roadway to either return to his original route or reach his destination via the alternate route. Typically, this is accomplished with the trailblazer assemblies and additional guide signs (if necessary).

In many cases, existing fixed signing will be adequate for the route guidance/route confirmation function, although it is expected that some new signs will have to be added for IMIS. Care must be taken to insure that there will be no conflict between new and existing sign messages which could confuse a motorist. If necessary, existing signs should be revised to eliminate the conflict.

Another type of fixed signing used in IMIS is referred to as "system identification" signing. The purpose of these signs is to inform the motorist of the existence and extent of IMIS (similar to those used for motorist aid call box systems). A typical message might be "Motorist Information System - Next 20 Miles" (32 kilometers). Consideration can also be given to including a "logo" in the

Table 35. Typical Cost Estimate for a Variable Message Signing Configuration*

(1) Capital Costs

Variable Message Signs (Disc Type)	
• Three-line	$48,000
• One-line insert	9,000
Fixed Message Signing	
• New Guide sign panel used with one-line VMS insert	2,500
Mounting Structures	
• Overhead sign bridge for three line sign (approx. span length 62 ft., 99 km) Cost includes structure, guard rail protection, and disc sign installation	38,200
• Incidentals for overhead structure (conduit, signal cabinet wire, pull boxes, exterior lighting, etc.)	3,500
• Cantilever structure for new guide sign panel and one line variable insert. Cost includes structure, guard rail, and sign installation	15,500
• Incidentals for cantilever structure (conduit, signal cabinet wire, pull boxes, exterior lighting, etc.)	1,800
Total	$118,500

(2) Operating Costs

Assumptions:

Variable Message Signs (disc type) will be operational for 24 hours.
Average of 2 message changes per hour of operation
Use four (4) 400 watt floodlights to illuminate three-line sign during nighttime operation. Two (2) 400 watt floodlight for the one line sign.

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*This configuration is illustrated in Figure 14

Table 35. Typical Cost Estimate for a Variable Message Signing Configuration (Continued)

Disc sign illumination used 12 hrs/day (12 hrs darkness)
Cost of electrical power = $0.06 per kwh

Three Line Sign

Power Consumption:

- Illumination = 4 x 400 watts = 1600 watts = 1.6 KW
- Message Change = 0.5 watts (negligible)

Calculation:

- Hourly Cost - (1.6KW) ($0.06/KWH) = $0.096/Hr approx $0.10/hr.
- Daily Cost = ($0.10/hr) (12 hrs) = $1.20/day
- Annual Cost = ($1.20/day) (365 day/hr) = $438/hr.

One Line Variable Insert

Power Consumption:

- Illumination = 2 x 400 watts = 800 watts = 0.8 KW

Calculation:

- Hourly Cost = (0.8KW) (0.06/KWH) = $0.048/hr approx. $0.05/hr.
- Daily cost = ($0.05/hr) (12 hrs) - $0.60/day
- Annual Cost = ($0.60/day) (365 days/yr) = $219/yr.

Total Annual Power Consumption For Variable Message Signs at Site

• Three-line sign	$438
• Panel insert sign	219
TOTAL	$657

(3) Maintenance Cost

- Annual Maintenance Cost (3 percent of capital cost of the sign itself)

Three-line sign	$1,440
Panel Insert Sign	270
TOTAL FOR SIGNING SITE	$1,710

signs. The logo could then be used on other signing (e.g. "special" route markings) throughout the corridor to provide an IMIS identity. The system identification signs should be placed at key entry points to the corridor and at appropriate exit points to denote the end of the system.

There will be other fixed signing in the corridor, associated with specific functions such as ramp metering, highway advisory radio, and motorist aid call boxes. These are treated as an integral part of the individual functions and are thus not included here.

12.4.2 Quantities and Unit Cost Estimates

A. Route Guidance/Route Confirmation Signing

The number of new route guidance/route confirmation signs required will depend on corridor size, number of alternate routes included and the extent to which such signing already exists. One method to obtain an estimate of the number and type of signs needed is by field survey; i.e. to drive through the corridor and follow the typical diversion routes, using maps to note the adequacy of existing signs and needs for additional ones. However, since this may be time consuming and since such fixed signing is not a major system cost item, a rough "lump sum" estimate based on general corridor familiarity will probably suffice. For example, one might assume some average cost, such as $500 per corridor mile ($311 per corridor kilometer). Then, if the corridor is about 20 miles (32 kilometers) long, the total lump sum cost for route guidance/route confirmation would be estimated at $10,000. Of course, some judgement is necessary in selecting such a unit price; however, as noted previously, total system cost is rather insensitive to this item. There would be no associated operating costs. Annual maintenance costs (e.g. repair or replacement of "knockdowns") are probably small enough to be considered negligible, but can be included if desired.

B. System Identification Signing

The number of system identification signs will depend primarily on the number of freeways serving as major entry points to the corridor and exit points from the corridor, for both directions of flow. Normally, only such points at the lengthwise ends of the corridor are signed (as opposed, for example, to a freeway entering at the corridor midpoint); however, other locations can be included if considered warranted.

Roadside mounting is the typical support configuration for this type of signing. Depending on freeway width, it may be necessary to install signs on both sides of the road to insure adequate exposure to the motorist. Where this is the case, a staggered arrangement is considered desirable.

The number of signs required can be simply estimated using a map of the corridor and a knowledge of each freeway's width (number of lanes) at the approach points. Average sign size should be on the order to 120 square feet

(11 square meters). Unit costs, installed, can be estimated from recent contract prices for similar type signing. An approximate cost of $1,000 per sign can probably be used as a rough estimate. There are no operating costs and annual maintenance should again be small enough to be neglected, but can be estimated if desired. Since the total cost for this type of signing will be relatively small, it can be treated as a "lump sum", obtained by multiplying the unit cost by the number of signs. This value can then be added to that of the route guidance/route confirmation type, resulting in the total cost for system fixed signing.

12.5 HIGHWAY ADVISORY RADIO

12.5.1 Introduction

Highway Advisory Radio (HAR), also referred to as "audio signing", is an additional or alternative method of providing motorists with real-time traffic information. Transmitters, located at roadside, broadcast information provided from the central control facility to a localized zone along the roadway, at frequencies which can be received on a motorist's standard AM radio. The FCC, under docket 20509 has allocated the frequencies 530 KHz and 1610 KHz. These frequencies are located just outside of the standard AM broadcast band and can be received by most AM radios. Possibly, the low frequency could be used for one direction of travel, and the higher frequency used for the opposing direction.

Currently HAR usage is expanding with additional research and evaluation underway. Results thus far have been encouraging, and it is presently considered a viable real-time motorist information technique. Furthermore, HAR has a substantial cost advantage over a large variable message sign and thereby warrants consideration as an alternative device particularly for lower cost system designs. Additional uses are to augment variable message signs in critical areas, or replace them for reasons other than cost (e.g., excessive density of visual signs, aesthetics, other physical factors).

Since the traffic control application of HAR is still relatively new, it is recommended that during the feasibility study, contact be made with the Traffic Systems Division, HRS-32, Office of Research, Federal Highway Administration, Washington, D.C. (Phone 703-557-5227) to obtain the latest information and experience on other installations to date.

12.5.2 Typical Subsystem Equipment

A typical roadside radio installation includes field equipment, central facility equipment and a communications medium from field to central. Since the communications medium for all IMIS functions is treated as a single subsystem, this will not be included in the present discussion.

The field equipment generally consists of the following:

- Roadside transmitter, including any necessary communications interface equipment

- Weatherproof cabinet and mounting provisions

- Antenna, either of the vertical monopole (whip) or cable radiator type. The cable antenna will provide a coverage zone of approximately 100 feet (30 meters) on each side of the cable for the entire cable length. A 6000 foot (1829 meter) cable will allow the motorist to receive two repetitions of a 40 second message, on a 55 mph (88.5 km/hr) highway.

- Associated visual signing, needed to alert the motorist to the upcoming radio information zone and provide radio tuning instructions (frequency on radio dial). It should be noted that if the roadside radio is used only at discrete times (as opposed to a 24 hours per day operation), it is desirable to include some device (e.g. beacons, blank-out sign) with the advance sign to inform the motorist as to when information is being transmitted. This will add to the "real-time" system credibility aspect, as well as alerting the motorist to the existence of the installation.

The related equipment at the central facility includes the following:

- Tape recorder unit, to record messages to be transmitted to the field. Note that one tape recorder unit can be used for several roadside installations.

- Tape playback unit, to play the recorded message for field transmission. Normally one playback unit is dedicated to each roadside installation.

- A supply of tape cartridges.

- Amplifier, power supply, switches and controls, depending on available off-the-shelf equipment and desired control/monitoring capabilities.

- Communications interface unit

- Rack or console to house the central equipment

12.5.3 Equipment Trade-Offs

The major trade-off related to roadside radio equipment is the type of antenna used, i.e., monopole or cable radiator. Primary trade-off factors are cost of installation and coverage zone provided.

The cable radiator runs the total length of the coverage zone. It is generally buried 6 to 8 inches (15 to 20 centimeters) below the surface in the median or along the roadside. The cable may also be hung on structures. As such, the cable radiator costs (material and installation) are substantially higher than the single monopole antenna installation. (An HAR system with a monopole antenna costs roughly one-third that of a cable radiator HAR system.) On the other hand, the cable provides a well defined radiation zone, i.e. along the

roadway, with broadcast signal dropping off sharply in the lateral direction. The monopole is omnidirectional. This may create interference problems due to its extensive radiation zone.

From a functional point of view the general consensus is that the radiation coverage aspect must dominate the selection and thus the cable radiator, despite its higher cost, is the normally selected antenna configuration for urban roadway installations. For other applications such as parks or rural areas, the monopole would probably be more suitable.

12.5.4 Unit Costs

A set of unit cost estimates for capital, maintenance, and operating costs are presented below. These costs are considered representative for typical roadside radio installations, and may be used for the purposes of the feasibility study. Should a cost refinement be desired, it is suggested that the using agency contact manufacturers and/or users of such systems.

A. Capital Costs

Based on recent system installations, the average cost per roadside radio installation is approximately $20,000, which includes all field and central equipment, installation and system test.

B. Maintenance Costs

A reasonable estimate for annual maintenance costs is in the range of 10 to 15% of the capital cost. Thus, as an average, maintenance costs may be assumed to be approximately $2,500 per year per installation.

C. Operating Costs

Operating costs for electrical power and miscellaneous supplies are estimated to be approximately $200 per year per installation. Costs for operating personnel (e.g. for monitoring system, taping messages) are included as part of the IMIS system central facility, and thus not listed separately here.

12.6 ENTRANCE RAMP CONTROL

12.6.1 Introduction

The purpose of entrance ramp control is basically to limit the number of vehicles entering a freeway at one or more points, so that the freeway demand remains below capacity. Vehicles seeking entry to the freeway may then either wait in a queue, or divert to an alternate route. In the latter case, they may either again seek entry to the freeway at some other ramp (usually downstream), or continue on the alternate route to their destination without returning to the freeway.

Ramp control may impose a disbenefit on the "controlled" vehicles; however, the vehicles on the freeway (which represent a far greater number) achieve substantial savings when flow breakdown is prevented. An additional proven benefit is the reduction in accidents due to smoother flow on the mainline and a reduction in turbulence in the ramp vicinity.

The two basic forms of entrance ramp control are ramp metering and ramp closure. The latter is a rather severe measure and should only be used in extreme cases. Although it may have isolated application in an IMIS corridor, it is not considered necessary to identify these in the feasibility study. Thus, only ramp metering is treated further.

In the following portions of this section, basic ramp metering concepts and alternatives are briefly discussed. Field equipment for a typical installation is then indicated, and a unit cost estimate is given. For the reader who is interested in further details on ramp metering, the following reference provides a comprehensive discussion on all aspects of the subject: "Guidelines for Design and Operation of Ramp Control Systems", NCHRP Project 3-22 Report, December 1975.

12.6.2 Ramp Metering Concepts

Ramp metering is usually classified as being one of the following three types:

- pretimed
- local responsive
- system responsive

Pretimed metering is characterized by the fact that it operates independent of mainline traffic conditions. Metering rates are based on historic data and may vary with time of day, day of week, etc., but are not directly changed in response to real-time variations on the mainline. Detectors are generally used on the ramp for initiating the metering cycle when a vehicle is present.

Both responsive types of metering are influenced by mainline traffic as determined from mainline detectors. In the local case, metering rates are determined in response to mainline conditions in the ramp vicinity only. Thus this form of metering operates as an isolated control for the associated local portion of the freeway. Since it can respond to real-time traffic variations, it is more efficient than pre-timed metering.

System responsive metering represents the next step in sophistication. Here, a series of ramps are treated as a "system", and individual metering rates are determined to optimize the mainline flow throughout the length covered. Thus, for example, downstream conditions can influence the rates used at one or more

upstream ramps. For system responsive metering, all relevent detector data is analyzed by a central computer, and individual metering rates are then established by an algorithm programmed in the computer. Obviously, then, a communications link is required between each ramp and the central computer. This is the major "price" paid for the added level of sophistication.

In an isolated trade-off study where ramp metering is the only function considered, it is expected that the benefit/cost ratio would decrease as each level of sophistication is added. This is because the simplest and least costly form of metering (pretimed) addresses the major part of the problem, i.e., the recurrent congestion which is more or less predictable. In IMIS, however, communications facilities and a central computer are integral parts of the system. Thus, the incremental costs for the most sophisticated form of ramp metering become small enough to warrant its use. This capability is therefore assumed for the feasibility study.

Another classification of ramp metering deals with the manner in which vehicles are released in each metering cycle. The following alternatives are possible:

- Single Vehicle, Single Lane
- Multiple Vehicle, Single Lane
- Single Vehicle, Two Lane

Single vehicle, single lane metering allows one vehicle at a time to be released in each signal cycle. This is by far the most common metering form used. It provides the smoothest merge operation and the highest reduction in accidents in the ramp vicinity. Generally, a maximum of 900 vph can be accommodated with this approach.

In multiple vehicle metering (also referred to as "platoon metering") the cycle time green phase is extended to permit additional vehicle to pass through per cycle. A practical limit is probably 3 vehicles per cycle. Higher overall metering rates are possible with this approach (about 1100 vph), but greater merging friction and higher accident potential are the concomitant disadvantages.

Two lane, single vehicle metering can provide metering rates comparable to platoon metering. In this case, the ramp geometry must be such as to allow for two lanes approaching the metering signal. Vehicles may be released simultaneously or in a staggered mode (one from one lane, then one from the other); in either case they must ultimately use a single lane for entry onto the freeway. An advantage of having two lanes on the ramp is that it provides greater storage capacity.

An additional form of ramp entry control consists of metering in one lane, while providing a second unmetered lane for buses and/or car pools. The purpose here is to give priority to high occupancy vehicles by allowing them to

bypass the ramp queue in the metered lane. This type of control is receiving increased consideration, since it is consistent with national energy conservation principles. It normally entails reconstruction of the ramp; however, the cost is often considered justified. Fairly extensive traffic engineering studies are usually performed to evaluate potential modal shifts (into buses or car pools) and determine the basic feasibility of this type of operation.

Of all the ramp metering alternatives noted above, the single vehicle, single lane approach is expected to be the dominant one in an IMIS corridor application. Therefore, this form is used for developing unit cost estimates. Field equipment for all of the approaches are generally similar and thus the cost estimate is considered sufficiently accurate even if one of the others is eventually considered for some isolated cases. One exception would be ramp reconstruction costs for priority bypass lanes. Some allowance for this can be made in the feasibility study if it has already been determined that corridor operations will include this feature; however, the associated costs should not be charged to IMIS. Otherwise, it is more appropriate to defer this type of treatment to the final design phase.

12.6.3 Typical Field Equipment Complement

While there are some variations depending on specific applications, the most typical complement of equipment used for a metered installation ramp is noted below:

- Loop Detectors - Three detectors are normally used for each metered ramp. A "demand" detector is located immediately upstream of the metering signal stop line; it is used to indicate the presence of a vehicle at the signal and initiate the metering cycle. A "passage" detector is located immediately downstream of the stop line; it is used to inhibit the next green signal indication until the previous vehicle has cleared the location. A "queue" detector is located upstream, either in the vicinity of the ramp entrance or on the surface street (depending on geometry and available storage length). Its function is to detect when the ramp queue is becoming excessively long so as to interfere with traffic operations on the surface street. When this occurs, metering rates are increased to shorten the queue.

  (Mainline detectors associated with responsive metering are considered as part of the freeway surveillance subsystem and are thus not included as part of the ramp metering installation).

- Ramp Signals - Two signal heads are used at each metered ramp. The most common configuration is to locate a pedestal-mounted signal head on each side of the roadway. (A single pedestal mount on the left side of the road with two signal heads at different heights has also been used). Two-color signal heads, i.e., red and green, are most common, although three-color heads have been used in some applications.

For the feasibility study, it is recommended that the cost estimating purposes, the two-pedestal, two-color signal head configuration be assumed.

- Ramp Controller - Since system responsive metering is envisioned in an IMIS corridor, a microprocessor type controller is considered most appropriate. This provides maximum control flexibility as well as more sophisticated back-up modes. Determination of optimum system-wide metering rates will normally be done at the central computer facility; however most other local data processing functions and associated logic can be performed by the microprocessor controller.

- Signing - An advanced warning sign is normally installed upstream of the metering signal to inform approaching motorists that the ramp is metered. Typically, two flashing beacons are used with the sign to indicate when metering is in effect. The sign is usually diamond-shaped and bears a legend such as "Ramp Metered When Flashing."

  Additional signing is normally used at the stop line to indicate specific instructions to the motorists on the metering procedure. Most often, one-car-at-a-time is the metering technique, and an appropriate legend so indicates (e.g., "One Car Per Green"). Most operational systems have also used a sign in conjunction with the stop line with typical legends such as "Stop Here On Red", or "Wait Here For Green" (with or without an arrow). These serve to insure against motorists stopping too far back to be detected by the "demand" detector. Signs such as the foregoing may be mounted on the signal pedestals.

12.6.4 Unit Cost Estimates

A. Capital

An estimated cost for a typical metered ramp installation is indicated below and includes equipment, installation and checkout. These may be refined by the using agency if more accurate cost data can be obtained.

• Detector Electronics (3)	$1,050
• Signals and Poles (2 each)	1,400
• Ramp Controller	2,000
• Installation of loops, loop lead-in and signal head wiring	3,000
• Installation of 500' (152 meters) of conduit for queue detector connection	4,000
• Ramp Signing	500
• Miscellaneous (loop and lead-in wire, conduit, pull box, etc.)	2,100
Total	$14,050

The above costs do not include cabinets or communications units, since these are treated separately elsewhere.

B. Annual Maintenance Cost

Annual Maintenance costs for equipment per metered ramp, is estimated as follows:

Three detectors @ $40	$120
Controller @ 10% of capital cost	200
Signal Heads @ $30	60
Total	$380; use $400

C. Annual Operating Cost

The only significant operating cost is for signal head power, which is estimated at $120/year for the pair of heads.

12.7 FREEWAY SURVEILLANCE

12.7.1 Introduction

This section deals with the IMIS automatic surveillance subsystem applicable to all freeways in the IMIS corridor. Ramp surveillance is considered to be part of this subsystem, except for metered entrance ramps which were treated separately in previous section (Entrance Ramp Control).

The function of the automatic surveillance subsystem is to acquire all of the real-time data required for system operation. Specifically, the data is used for the following:

- Determination of existing traffic conditions
- Short term predictions of variations from present traffic conditions
- Automatic detection of incidents
- Implementation of appropriate control strategies
- System evaluation by means of various on-line measures of effectiveness
- Development of historic data base for subsequent use in updating system parameters

In addition, although not a direct objective of the surveillance subsystem, an extensive area-wide network of permanent counting stations will be available

for providing various categories of traffic data (volume, occupancy, speed, classification, and queue lengths) on a continuous basis, and for any averaging period which is desired. Such data will be useful for many other studies and for planning purposes.

12.7.2 Trade-off Considerations

There are two basic trade-off considerations for the automatic surveillance subsystem; the type of detector to be used, and the spacing between detector stations, These are discussed in the following paragraphs.

A. Detector Type

Within the present state of the art, there are three types of detectors considered suitable for freeway surveillance use. These are:

- Sonic
- Magnetic
- Inductive Loop

Pertinent characteristics of each of these detector types, including measurement capability, principle of operation, advantages, and disadvantages, are summarized in Table 36.*

There is general agreement throughout the traffic engineering community that the inductive loop detector is the best overall choice for the freeway surveillance application. It is the most commonly used type, and considered to be the most accurate, most flexible, and most reliable for the present state of the art. Use of inductive loop detectors is therefore assumed for developing surveillance cost estimates in the feasibility study.

B. Detector Spacing

The major factor that influences the spacing of detector stations** along the freeway mainline is the automatic incident detection function. Incident detection algorithms are generally based on a comparison of data from a contiguous set of detector stations to establish whether an unusual flow pattern exists. For example, under normal flow conditions two adjacent stations would indicate compatible traffic parameters, such as volume or occupancy. (Intervening exit or entrance ramps are accounted for between detector stations). When an incident

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*This table is a reproduction of part of Table 16 contained in "Urban Freeway Surveillance and Control - The State of the Art", U.S. Dept. of Transportation, Federal Highway Administration, Revised Edition, June 1973 (Authored by P.F. Everall).

**A detector station represents a location on a given roadway where one or more lanes are instrumented with detectors. A station corresponds to one flow direction only.

Table 36. Detector Characteristics

Detector	Measurement capability					Principle of operation	Advantages	Disadvantages
	Passage count	Presence	Speed	Occupancy	Queue length			
Sonic: (a) Pulsed	x	x	x	x	x	Emits bursts of 20 kHz at 20—25 times per second. Vehicle reduces path length, resulting in the return signal arriving when the receiver is open. Can be overhead or side mounted.	1. No disturbance to road and a minimum disturbance to traffic for its installation. 2. Can be used in locations with unstable pavement. 3. Does not produce electromagnetic raidation. 4. Can classify vehicles by height.	1. Can be expensive to install if no suitable pole or mounting available. 2. Inaccurate due to conical detection zone, and due to wide variations in vehicle configurations and height. 3. Insensitive to vehicle direction. 4. Sensitive to environmental conditions, dirt, traffic, vibrations, etc. 5. Inaccurate under congested conditions. 6. Sonic output can be annoying to humans or animals if not adjusted properly.
(b) Continuous wave	x	-----	x	----	-----	Operates on the Doppler principle.	1. As 1, 2, and 3 above. 2. Improved accuracy for speed measurement.	1. As 1, 4, 5, and 6 above. 2. Expensive for limited amount of information. 3. Is not a presence detector, and cannot detect stopped vehicles (3 mph).* *1 mph = 1.61 km/h

Table 36. Detector Characteristics (Continued)

Detector	Measurement capability					Principle of operation	Advantages	Disadvantages
	Passage/count	Presence	Speed	Occupancy	Queue length			
Inductive loop: (a) Fixed tuned	x	x	x	x	x	Vehicle passage cuts the magnetic flux around a resonahtly turned loop (at 100 kHz) thereby increasing or decreasing the inductance so that a change in resonant frequency, impedance, amplitude or phase shift is detected and transmitted to amplifying or relaying circuit. 1-4 turns of insulated wire installed in slot in pavement and covered by hot asphalt mix.	1. Size and shape of detection zone is adjustable (necessary for queue detector). 2. Excellent presence detector. 3. Relatively insensitive to weather conditions.	1. Cost of installation can be excessive. 2. Traffic disruption during installation. 3. Difficult to tune so that both motorcycles and highbed vehicles are counted.
(b) Self-tuning	x	x	x	x	x	As above - - - - - - - - - - - - - -	As above, but in addition: 1. No initial or periodic calibration required.	As 1-3, above, but in addition: 1. A number of self-tuning detectors located in the same controller cabinet have been known to interfere with one another, indicating importance of case shielding and cable coupling. 2. Response time may be slower than with fixed tuned ones.

Table 36. Detector Characteristics (Continued)

Detector	Measurement capability					Principle of operation	Advantages	Disadvantages
	Passage count	Presence	Speed	Occupancy	Queue length			
Magnetic:								
(a) Magnetometer	x	x	-----	x	-----	Passage of vehicle disturbs vertical component of earth's magnetic field, thereby inducing a voltage in a coil placed beneath the pavement.	1. Relatively inexpensive to install. 2. Unaffected by weather conditions. 3. Does not produce any radiation.	1. Poorly defined detection zone that may necessitate more than one sensing head. 2. Will double count some vehicles due to magnetic material distribution within the vehicle (or may miss close-following vehicles if adjusted for this). 3. Subject to electromagnetic interference if not compensated. 4. Are not very suitable where ferrous material is present (e.g., bridge decks).
(b) Magnetic coupling	x	x	-----	-----	-----	Vehicle passage disturbs magnetic field generated by emitter coil and is sensed by a second orthogonal coil.	1. May be used to obtain vehicle classifications.	1. May be unstable due to instability of orthogonal coilds in pavement surface.

occurs, the upstream station will show an increase (with time) in the parameter values, while the downstream station will show the reverse. The rapidity of the change will be a function of the existing volume levels.

It is apparent, then, that the closer the detector stations are spaced, the more rapid the incident can be detected. There is a limit, however, since a finite amount of time is required for data smoothing and minimization of "false alarms".

Automatic incident detection provides a major system benefit, since the earlier the incident is detected, the more rapidly it can be serviced and cleared, and flow returned to normal. Since delay is proportional to the square of the detection time, reductions in the latter are very effective in generating benefits. Furthermore, the sooner the incident is detected, the sooner system control can be applied (e.g., diversion of traffic to an alternate route, ramp metering, etc.) and thus additional major benefits accrued. Other benefits of early detection include reduction of secondary accidents (by warning oncoming traffic) and more rapid medical aid for any injured people.

Determining costs versus detector spacing is a relatively straightforward task. However, benefit determination involves a very complex analysis if one is to properly account for the "real world" environment that the system operates in. For example, for given detector spacings and state of the art algorithms, one can estimate the performance (detection time with a stated "false alarm" rate) for each spacing. However, the incident could be detected first by other means (e.g., a patrolling police car, a patrolling helicopter, a motorist using a roadside callbox, CB radio, etc.). First detection by each of these other means has some probability of occurrence; unfortunately it is difficult to estimate these probabilities without an extensive data base for each. Thus, it is difficult to truly assess the advantages (benefits) of one spacing versus another in a general case.

Detailed studies, using both analytical and computer simulation techniques for benefit determination were performed during the IMIS feasibility study for the Northern Long Island Corridor. The study examined detector spacings of 1 mile (1.6 km), 1/2 mile (0.8 km), and 1/4 mile (0.4 km), using an incremental benefit/cost ratio approach. It was found that the incremental benefit/cost ratio in going from 1 mile (1.6 km) spacing to 1/2 mile (0.8 km) spacing was 1.7, but in going from 1/2 mile (0.8 km) to 1/4 mile (0.4 km) the incremental benefit/cost ratio dropped to 0.9. Thus, 1/2 mile (0.8 km) spacing was recommended. While the analysis was site-specific, the results are compatible with the most commonly used detector spacing in freeway surveillance systems, i.e., 1/2 mile (0.8 km).

Because of the complexity involved in this type of analysis, it is considered appropriate to accept the 1/2 mile (0.8 km) spacing for use in the feasibility study.

A related issue is whether or not to install detectors in every lane at a given station. Experience shows that the present trend is to instrument all lanes. One reason is that the incremental costs are not large, once a station is being constructed. A second reason is that full instrumentation provides flexibility to incorporate improved incident detection capability as the state of the art in detection algorithms advances. Thus, for the feasibility study, it is recommended that all lanes be considered instrumented.

Finally, it is usual to include double detector stations (sometimes referred to as a "trap" configuration) every so often , typically at 5 mile (8 km) intervals. These serve to provide accurate speed and classification measurement for system use. These, too, are assumed for the study.

12.7.3 Unit Cost Estimates

Unit cost estimates should be developed on a lane basis. Before developing the estimates, the freeway configuration should be examined to determine the number of lanes in the various sections. If one configuration predominates it may be used as "typical". If not, unit cost estimates can be developed for each configuration. In this case, the number of freeway miles associated with each configuration should be recorded.

If cost data are available (e.g. from recent contracts) they should be used for the unit estimates. As an alternative the following typical costs are provided:

A. Unit Capital Costs

Basic mainline detector station	\$1000/lane
Add for double detector station	800/lane
Unmetered ramp (entrance or exit)	1,100/ramp

The above capital costs include all hardware, material, installation, and checkout. Communications and cabinet costs are not included since they are treated separately elsewhere.

B. Unit Maintenance Costs

Estimated annual maintenance cost are as follows:

Basic mainline detector station	\$40/lane/year
Add for double detector station	\$40/lane/year
Unmetered ramp (entrance or exit)	\$40/ramp/year

C. Unit Operating Costs

Annual operating costs are those associated with power for the detector electronics. Since the power requirements are very small (less than 10 watts), it is probably acceptable to neglect these costs. However, they may be estimated, if desired, from the expression:

Unit Annual Op. Cost = 8760 hrs/yr x .010 kilowatts x cost/kilowatt-hr

= 87.6 kilowatt-hrs/yr x cost/kilowatt-hr.

12.8 ARTERIAL SURVEILLANCE AND CONTROL

12.8.1 Introduction

In IMIS, signals on the candidate arterials are brough under some form of central computer control. The primary reason is to maximize the available excess capacity on the arterial under conditions when diversion to the arterial is required. Once implemented for this purpose, however, the arterial control subsystem becomes available for use at all time periods and thus generates significant additional system benefits.

Arterial surveillance is necessary to provide the real-time traffic data required for both responsive arterial signal control and overall system control. It is assumed for the feasibility study that the inductive loop detector is the type used to provide the surveillance data for the same reasons as noted in the freeway case.

12.8.2 Arterial Control

For IMIS, two basic approaches may be considered for establishing computer control on the candidate arterials. These are:

- Addition of new interconnected traffic signal groups and extensions of existing interconnected groups, with communications to the central computer from each master controller (master/central approach).

- Communication to the central computer from all traffic signal controllers except where communication with the master controller of an interconnected group provides satisfactory control at lower cost (central control approach).

The two approaches differ primarily in the type of control that may be exercised by the central computer. With the master/central approach, the number of signal timing patterns is limited to those that are available in the master controllers, while with the central approach a theoretically unlimited number of signal timing patterns are available. The master/central approach requires connection to the communication medium only at each master controller, while the central approach requires connection to the communication medium at

all controllers except where a master-controller provides satisfactory timing patterns. For both approaches, vehicle detectors (and any variable message signs) along the arterial must also be connected to the communication medium. Both approaches can provide locally coordinated signal timing during standby periods (central computer/communications not operating).

From the point of view of system performance, the added flexibility of the central control approach is obviously desirable. From an overall viewpoint, however, including cost considerations, it is difficult to provide a general recommendation as to which of the two approaches should be adopted since there are site-specific factors which can significantly influence the decision. Such factors include:

- The extent of interconnect that presently exists.
- The sophistication of the level of control provided by existing masters.
- The closeness of the arterials to the freeways and thus the ability to tie in to the freeway communications facilities. (A good example is a service road.)

In order to avoid a prolonged trade-off analysis, it is recommended that the central control approach be assumed in the feasibility study for cost estimating purposes. This may produce a conservative result if a significant amount of interconnect already exists; however, the difference is not expected to be substantial since (1) provisions must be made in either case to bring detector data back to central, and (2) the cost of controller/communications interface units for the central control approach is relatively small. Costs for the two approaches are expected to be relatively close when little interconnect exists because of the need to add master controllers for the master/central approach.

12.8.3 Critical Intersection Control

In many cases, certain key intersections will experience heavy short-term demands and rapid demand fluctuations. In computerized signal systems, such intersections are best operated with a form of control termed "critical intersection control" (CIC).

CIC operation is imposed on a cycle-by-cycle basis at the key intersections within a control area subsection while still maintaining the basic traffic control pattern required throughout the subsection. This is accomplished by varying the split at the selected intersections in accordance with local traffic demands, but at the same time, preserving the original cycle and offset. In this way, individual intersections with heavy and variable traffic volumes can be made to react to cycle demands as they occur, so that normal traffic flows in the subsection are not disrupted by blockages at these critical intersections. This combination of local intersection control in conjunction with a background pattern covering a group of

intersections provides for the most efficient movement of traffic in the subsection. Volume thresholds and minimum "green" limits are included in typical CIC algorithms to assure that reasonable volume levels exist during CIC operation and that all traffic movements proceed with safety.

Selection of intersections for CIC treatment can be accomplished in two basic steps:

- Identification of intersection which are potential CIC candidates

- Determination of which of the potential candidates warrant operation as CIC's

Various levels of analysis may be used in each of the above steps, ranging from judgment based on a knowledge of operating conditions, to in-depth analysis based on an extensive data base.

Since this is a feasibility study, a reaonsable estimate of the number of locations requiring CIC treatment is adequate. It is felt, therefore, that certainly the first step, and probably the second, can be based on judgment in conjunction with discussions with operating personnel most familiar with arterial signal operations. However, in the event that a more detailed study is considered necessary, a more formal procedure to select the CIC's is provided in Appendix D. The appendix defines the associated data collection requirements and provides "warrants" for designating candidate intersections for critical intersection control.

The most typical configuration for surveillance associated with CIC's is to instrument each lane of each approach with detectors. While there can be exceptions, the additional data collection and analysis required to make this determination are not considered warranted for the feasibility study. Therefore, it is recommended that the all-lane all-approach configuration be presently assumed.

12.8.4 Arterial Surveillance (Non-CIC)

The typical surveillance configuration for arterials is to locate detectors up-stream of signalized intersections. When intersection spacing is large, intermediate detector stations are added to fill in major gaps in the surveillance coverage.

The number of arterial detector stations required (NDS_A) may be estimated from the following:

- If the average spacing between signalized intersections is equal to or less than 1 mile (1.6 km),

$$NDS_A = 2\,(NSI - NCIC) \tag{6}$$

where NSI = The number of signalized intersections on the arterial. (The factor of 2 is to account for both flow directions, since each is considered as a separate station).

NCIC = The number of "critical intersections"

- If the average spacing between intersections is significantly greater than 1 mile (1.6 km),

NDS_A = 1 per mile (per 1.6 km) per roadway direction, less the number of "critical intersections" (each critical intersection applies to both roadway directions).

For a given detector station, partial lane coverage may be adequate in some cases (with multiplying factors applied to expand the count data to the full width), while full lane coverage may be advisable in other cases. A reliable determination of the adequacy of partial lane coverage involves field data collection at each site (e.g., volume counts by lane during typical times of day and typical days of the week). To avoid this fairly extensive data collection and analyses effort, it is suggested that full lane coverage be assumed in the feasibility study for cost estimating purposes. Since additional detectors are not a major cost item for arterial surveillance and control (major cost item is associated with trenching along the arterial), this assumption should not have a substantial impact on system cost.

12.8.5 Unit Costs

Costs for arterial surveillance are dependent on the communication medium used. This dependence is due to the need to connect the controllers and detectors to a telemetry cabinet. If an owned cable along the arterial is used as the communications medium, a major portion of the trenching costs are attributed to the communications subsystem. If other than owned-cable is used, the full cost of arterial trenching must be borne by the arterial surveillance and control function.

If the user chooses to calculate his own unit costs for this function, it is suggested that the section on communications be first reviewed, so that a judgment as to which communications medium is desired can be made. In this way, it will not be necessary to develop two sets of cost data.

Alternatively, the user may choose to use the handbook estimates. For this case, the two sets of arterial surveillance cost data are provided in Table 37. The table also includes the costs associated with the control function.

12.9 OTHER SYSTEM SURVEILLANCE

12.9.1 Introduction

Automatic electronic surveillance is a requirement and thus an inherent part of IMIS. This, however, does not preclude the possible use of other manual surveillance techniques as an adjunct to the automatic system. Indeed, at least some form of manual surveillance must be in current use in the corridor.

Table 37. Typical Arterial Surveillance and Control Unit Costs

	OWNED CABLE		OTHER MEDIA	
	PER CIC*	PER NON-CIC*	PER CIC*	PER NON-CIC*
A. ARTERIAL SURVEILLANCE (1)				
• Capital Cost	$27,500	$2,500	$45,500	$11,500
• Maintenance Cost	400	100	400	100
• Operating Cost	90	20	90	20
B. ARTERIAL CONTROL (2)				
• Capital Cost	$500/intersection			
• Maintenance	$50/intersection			
• Operating Cost	(Negligible)			

(1) Costs are average of two-lane and three-lane (per direction) arterials cases. Difference for either is within 5% for capital costs. Differences for maintenance and operations costs may be neglected.

(2) Assumes central control approach and use of existing controllers. Costs are for controller/communications interface units. Communications and cabinet costs are treated separately elsewhere.

*Each CIC cost includes all approaches. Non-CIC costs are per detector station for one flow direction.

This section discusses several of the alternatives, including closed circuit television, ground patrolling vehicles, helicopters, and citizen's band radio. For the most part, these techniques are associated with the incident detection or incident verification function, and are therefore treated within that framework.

12.9.2 Closed Circuit Television (CCTV)

Automatic electronic surveillance represents the backbone of the surveillance subsystem, yet it is effectively "blind" with regard to type of incident which it detects. Thus, visual verification is required before the necessary services (tow truck, ambulance, fire fighting equipment, etc.) can be dispatched. Normally, an observer at the scene (usually police) must make the appraisal. The time that elapses between occurrence and appraisal affects the amount of delay encountered by motorists and can, of course, be critical to the person involved. Closed circuit television can conceivably close this time gap through remote visual monitoring.

Typically, for such applications, CCTV would be used in conjunction with the automatic incident detection system, the latter providing the initial alarm. The operator would then observe the appropriate monitor to verify the condition and then take appropriate action. (The alternative of having continuous monitoring of all TV screens is simply not practical except when a minimal number of cameras are used.)

It is evident from the outset that for an IMIS corridor, complete CCTV coverage is not feasible from a benefit/cost point of view. Such coverage would entail a multi-million dollar capital investment plus high annual operating and maintenance costs. The incremental benefits through reduced response time over that provided by the automatic surveillance system (in conjunction with other verification sources such as patrolling police vehicles or motorist aid phones) cannot approach the costs.

The next consideration, then, is the use of CCTV at isolated locations or over limited sections of roadway. (High accident areas would be obvious candidates.) However, even if a limited number of cameras are used throughout the corridor, wideband communication facilities (coaxial cable or microwave) would become necessary to accommodate the signal bandwidth. Since such facilities are not required for any other IMIS function, the added cost would be attributed solely to the TV. Benefits achievable through application of TV to limited sections or isolated locations would not be substantial since they would accrue from only a very small fraction of the incidents occurring in the corridor.

In general, therefore, it is concluded that CCTV cannot be justified on a benefit/cost basis in an IMIS project. It is recognized, however, that there may be special circumstances in a given corridor which, in the judgment of the operating agency, warrant inclusion of some television coverage. In the event that this be the case, approximate unit costs are provided below:

Item	Capital Cost	Annual Maintenance	Annual Operating
Field Camera Site			
Camera Unit, complete	$14,000	$1,400	$50
Installation Services	400		
Mounting Pole	4,000	200	
Control Cabinet	700	(negl)	(negl)
Totals	$19,100	$1,600	$50
Central Control and Monitor			
Camera Control Unit	$10,000	$1,000	
Control Signal Modulators	500	50	
TV Receivers	500	50	$500
Video Recorder & Monitor	2,000	200	
Miscellaneous Wiring	500	(negl)	
Totals	$13,500	$1,300	$500
Grand Totals	$32,600	$2,900	$550

It should also be remembered that when estimating the IMIS communications subsystem, wideband facilities must be used for the communications medium if television is included. (The communications subsystem is discussed in Chapter 13.)

12.9.3 Ground Patrolling Vehicles

It is common policy for police jurisdictions to patrol limited access facilities, both for law enforcement and motorist aid purposes. The number of vehicles used and the patrolling frequency varies with numerous factors, the major one being available resources. It is obvious that the more vehicles used, the more rapidly an incident can be detected and serviced. Determining cost effectiveness, however, is a very difficult problem to generalize, and particularly when considered in conjunction with other IMIS capabilities (e.g. automatic incident detection, motorist aid phones). It entails consideration of probabilistic models for response times, number of vehicles presently used, and policy aspects. The latter affects overall incident duration in that in some cases police vehicles have "push bumpers" and can therefore remove some percentage of disabled vehicles from the active lanes, while in other cases police vehicles are not permitted to move vehicles, usually due to legal considerations. In light of such complexity, the problem of assessing the benefits of adding one or more police vehicles to the number already existing is indeed a difficult one.

A similar problem exists when considering the addition of patrolling tow trucks. Generally, adding one tow truck would be more effective than adding one police car, because the tow truck has the greater capability to remove vehicles

from the travelled way and thus minimize incident effects and duration. (Admittedly, police have the greater capability for overall incident management; however, improvements in tow truck response times generally produce the larger benefit.)

Since rigorous analysis is difficult for this subject, it is suggested that the matter be treated more qualitatively for the feasibility study. The using agency should assess typical response times for incidents through operating experience and/or discussions with police. If the times appear unduly large or the patrolling force obviously too scant for the miles covered, one or more additional police vehicles can be recommended.

Patrolling trucks are usually cost effective and should be considered if not presently used. Two options are available, i.e., owned or franchised. In the case of owned, the cost is borne by the using agency. Depending on operating policy, use of these trucks may be limited to removing vehicles from the limited access facility, after which a private franchised operator would remove the vehicle to a repair garage.

Franchised patrolling tow trucks can yield benefits to both the State and the motorist. In addition to not bearing the operating costs, the State may in fact realize an income from franchising. Motorists also benefit because the State is in a position to monitor tow operator performance including response times, fees, and repair charges. It appears, then, that a franchising arrangement is preferable to state-owned tow trucks.

The decision to include additional ground patrolling vehicles is left to the judgment and experience of the using agency. If included, the total equivalent cost per vehicle (including vehicle operator) may be estimated by the agency, or the following values used as an alternative:

Patrol car	$30,000/year
Tow Truck (owned)	$35,000/year
Tow Truck (franchised)	No cost

12.9.4 Helicopters

In large metropolitan areas it is not uncommon to have helicopter traffic surveillance during the peak rush hours. The organizations providing the coverage are typically (1) the large commercial radio stations, as a service to their listeners, and (2) the local police jurisdictions. In the latter case, police helicopters are usually not dedicated to the traffic surveillance function, but may be used routinely for this purpose when available.

If such coverage already exists in the IMIS corridor, the utility of the helicopters can be increased by coordinating their operations with the IMIS control center. Incident alarms from the automatic surveillance system (or other sources) can be relayed to the helicopter, which can then rapidly reach the location and notify the cognizant organization of the type of service required (ambulance, fire

fighting equipment, etc.). Cost for an appropriate communications interface at the control center is minimal and considered warranted for the application.

If there is no existing helicopter coverage, the question is whether or not to add this function to IMIS. Analyses performed in the Northern Long Island IMIS study revealed that in the presence of the automatic incident detection system, the benefits of adding a helicopter were substantially less than the high annual operating costs. A major reason for this is that the helicopter can only shorten the time required to identify the needed service - it cannot actually perform the service. Thus additional time is still needed for the responding agency to arrive and service the incident. Ground patrolling vehicles (police cars, tow trucks) are considerably more cost effective since they are much less expensive and can usually begin incident servicing upon their arrival.

In summary, if helicopter surveillance exists in the corridor, its operations should be coordinated with the IMIS control center. Addition of new helicopters, dedicated solely to traffic surveillance, is not considered cost-effective as an adjunct to IMIS.

12.9.5 Citizens Band Radio

It is estimated that approximately one out of every 10 vehicles is presently equipped with a Citizens Band (CB) transceiver. Despite all the "chatter" on the CB channels, it has been demonstrated that much valuable information related to incident detection and management is being transmitted by CB operators. Various volunteer organization such as REACT (Radio Emergency Associated Citizens Teams) have emerged, and are serving to monitor the FCC-designated emergency frequency channel (Channel 9). They, in turn, notify the cognizant official organizations (in most cases the police) who then respond to the problem. Furthermore, direct monitoring of CB channels by the police has been increasing throughout the country.

The basic question at hand is the role of CB in IMIS. Should the IMIS control center monitor Channel 9 and/or another channel or channels used locally by CB operators for traffic information on particular roadways? Should the IMIS control center transmit traffic information in response to CB operator requests?

Despite the possibility of obtaining potentially useful information via direct monitoring, there are several negative aspects. Among them are:

- In densely populated areas, the channels are grossly overcrowded. Noise levels are high and transmissions are often incomplete or garbled due to CB operators "stepping on" each other (transmitting at the same time).

- Information is often inaccurate. There is also a tendency for CB operators to exaggerate a situation.

- Additional staffing would be requied at the control center. It is questionable as to whether benefits would warrant the costs.

The IMIS control center will have telephone communictions with the local police jurisdictions. Since the police usually obtain pertinent CB information through either direct monitoring or from volunteer groups, any important developments can be made known to the control center by the police. Therefore, a direct interface between IMIS and CB operators is felt not to be necessary. Furthermore, IMIS provides motorist information via variable message signs, roadside radio, and telephone. These should be recognized as the authoritative sources of information. The need for individual transmission to CB operators (who still represent only a small fraction of the motoring public) is therefore not considered warranted.

There is, however, a potential use of CB which is currently being evaluated in the Chicago area (Chicago Area Expressway Surveillance Project, Illinois DOT). This consists of installing CB receivers at intervals of several miles along the freeway with a communication link back to the central facility. When an incident is detected by the automatic surveillance system, the nearest receiver is energized to "listen in" on the appropriate channel. Normally, the mobile CB users in the area will be discussing any problem and providing each other with information on the location, type and severity of the incident, lanes open, length of backup, etc. This information could be useful at the control center. Because this approach is still in the evaluation stage, it is premature to recommend its inclusion in IMIS at this time. If the evaluation should produce positive results, however, it is suggested that this feature be incorporated in the IMIS design.

12.10 MOTORIST AID CALLBOX SUBSYSTEM

12.10.1 Introduction

The basic objective of a motorist aid callbox system (in this case a subsystem of IMIS) is to provide a means for making the needs of a disabled motorist known to an agency capable of responding to those needs. The benefits of such a system, which accrue through more rapid detection, response, and removal of problems may take any or all of the forms:

- Direct benefit to the disabled motorist, in any of the multitude of physical, and/or emotional ways
- Reduction of secondary accidents
- Delay savings for other motorist using the facility

Because of the difficulties involved in assigning dollar values to many of the benefits, justification for installing a motorist aid system in terms of benefit/cost ratio is rarely if ever done. Instead, justification is established in a more qualitative sense such as fulfilling a defined motorist need and improving the overall quality of travel. In any event, the IMIS concept enhances the ability to justify a motorist aid callbox system, since a major cost element - the communication medium - will be an integral part of IMIS for other functions. Thus as a subsystem of IMIS, motorist aid becomes more cost effective than it would be as a "stand alone" system.

In the event that a motorist aid callbox system already exists in the corridor, it may be treated in either of two ways: (1) it may be retained as is, and in this case, a communication link should be provided between the responding agency and the IMIS central control facility, or (2) it may be modified so that the IMIS communications medium is used instead of its present medium, if this will result in a significant cost saving (e.g., if currently a leased system and IMIS is to be an owned cable, leasing costs can be eliminated).

For the remainder of this section, it is presumed that there is no existing motorist aid system. Thus, discussions are included on relevent design aspects, trade-off factors, and unit cost estimates, to the extent necessary for the IMIS feasibility study. Should the reader desire more detailed information, the following Federal Highway Administration report is an excellent reference on the subject: "Motorist Aid Systems - State of the Art Report", Report Number FHWA-RD-IP-76-11, September 1976, (also available through NTIS as report number PB 264 774). The report covers all motorist aid system aspects, from planning, through design, installation, and evaluation. It also includes experience to date for existing systems and identifies major equipment suppliers and key features of their product lines.

12.10.2 Design Considerations

The FHWA has developed design requirements and guidelines for motorist aid systems. These are contained in FHWA Instructional Memorandum 20-1-72, dated June 16, 1972 (supercedes IM 20-1-20), The Federal Aid Highway Program Manual (Vol. 6, Chapter 8, Sec. 3, Subsec 3), and are also summarized in the previously referenced State-of-the-Art Report. The following aspects are specifically relevent to the feasibility study:

- Roadside call terminals normally shall not be spaced less than 1/2 mile (0.8 km) apart. Experience indicates that usage of a system decreases with increased spacing and that approximate 1 mile (1.6 km) spacings are a reasonable compromise between economic considerations and motorist trepidations.

- Roadside call terminals shall be placed on both sides of the highway at each location to discourage the motorist from crossing the highway.

- Interior illumination of the roadside call terminal should be provided when ambient light is not sufficient. Exterior lighting is permissible if needed during darkness.

- Signs should be placed at the beginning of the motorist aid system and at intermittent locations to inform the motorist of the existence of and length of the system. A sign should also be placed at the end of the system to so inform the motorist.

The first two of the above determine the number of call boxes in the system and thus the major cost element. The third represents an operational cost item which should be considered when there is a substantial number of callboxes in the system and external illumination is used. The last represents a capital cost element which should be included.

An additional cost item is the central facility equipment, which includes an operator's console and system related electronics for signal processing and logic functions. In most cases, response to the callbox is provided by an appropriate police jurisdiction; thus, a set of central equipment would be located in each responding organization's facility. A monitor console can be provided for the IMIS control center if desired. If a police representative (dispatcher) is stationed in the IMIS control center, the equipment would, of course, be located at the IMIS center.

The motorist aid callbox system will utilize the IMIS "backbone" communications network along the freeway; however, for connection from the freeway system to the local police jurisdictions, additional communication facilities will be required (except where the police barracks are very close to the freeway). It is expected that leased channels would be the most cost-effective communications medium for this purpose.

12.10.3 Trade-Off Considerations

There are two basic trade-off considerations related to a motorist aid callbox system, i.e., the communication medium and the communication mode.

For the communications medium there are two basic options available, i.e., wire or radio. Under present federal funding policies for motorist aid systems, it is required that the project specifications leave such choices open for competitive bidding, i.e., the medium cannot be specified by the State. However, in IMIS a different situation exists, wherein the communications medium must be chosen to satisfy a wide variety of system functions. The communications medium is therefore determined after all individual subsystem requirements are identified, so as to best accommodate the total needs of the system. Since the IMIS concept is geared toward cost-effectiveness through common use of facilities, particularly communications, this approach is consistent with the intent of the federal funding policy for motorist aid systems. The communications medium, then, need not be treated as a trade-off issue, per se, for the motorist aid subsystem.

Regarding the communication mode, this may be either two-way voice or coded message (usually via pushbutton to indicate type of service needed) as specified by the State, either being eligible for federal funding. There appears to be an overall preference for the two-way voice communications from both the motorist and responding agency points of view; however, it still remains a somewhat controversial subject. A rather complete discussion of the related issues excepted from the previously referenced State-of-the-Art Report has been reproduced in Appendix E.

The present objective for the feasibility study is to estimate a unit cost for the callbox, including installation. Thus, selection of the communication mode at this stage is important only to the extent that it affects the unit cost. The unit cost will also be somewhat dependent on the communications medium (wire or radio) since different equipment is used in each case. Past experience has indicated, however, that the costs for either type of callbox are of the same magnitude. It is suggested, therefore, that this trade-off (or selection based on preference) be left for the final design phase, and for the feasibility study, a representative average cost be used instead.

12.10.4 Unit Cost Estimates

The best sources for typical unit cost estimates are the motorist aid equipment manufacturers or system suppliers. If, for any reason such sources are not utilized, the following estimates may be used in the feasibility study. The estimates tend to be on the conservative side, but should be accurate to within ±20 percent.

A. Capital Costs:

Installed Callbox	- $1,500 each
Installed Central Control Equipment	- $30,000 to $40,000 for each facility
Installed Central Monitor Equipment	- $15,000 to $20,000 each

B. Maintenance Costs:

Use 10 percent of each capital cost for the annual maintenance cost. For the above values, the maintenance costs are then:

Call box	- $150 each per year
Central Control	- $3000 to $4000 each per year
Central Monitor	- $1500 to $2000 each per year

C. Operating Costs:

Operating costs for personnel are normally not specifically charged to the motorist aid subsystem of IMIS since only a small part-time effort should be required. This is illustrated by the following typical operating experience in terms of number of calls handled per day:

System	No. of Callboxes	Avg. Callbox Spacing (Miles)	ADT	Avg. Calls Per Day
I-80(Illinois)	302	1.0 - 1.5	20.000	25-30
I-95, I-195 (Florida)	90	0.5	165,000	35
I-91, I-84 (Conn.)	178	0.5	31,000-64,000	40

Note: 1 mile=1.6 km

With such relatively low activity, it is presumed that each agency (police dispatcher for response, IMIS staff member of monitoring) can adequately handle the associated workload with their existing staffs.

Operating cost for power for internal terminal lighting and external terminal illumination (if used) may be estimated in a relatively straightforward manner. For example, if each box required a total of 100 watts, the annual cost for electricity per box (assuming 12 hrs. of lighting per day and \$.10 per kilowatt-hour) would be

$$\text{Cost} = 12 \text{ hrs/day} \times 365 \text{ days/yr} \times 0.1 \text{ kilowatts} \times (.1 \text{ dollars/killowatt-hour})$$

$$= \$43.80 \text{ per year per callbox}$$

An additional operating cost is that associated with the leased lines required to connect the callboxes between the freeway communications system and the responding police jurisdiction(s), if applicable. To estimate of the number of channels required, determine the number of callboxes within each police jurisdiction and divide by 20, the latter representing the approximate number of callboxes which can be accommodated on each full duplex leased channel. Then, sum the number of channels required for each jurisdiction to arrive at the total requirement. Leasing costs should then be determined in conjunction with the local telephone company.

A final cost element to include is that of the signing specifically associated with the motorist aid system. Signing costs may be determined by multiplying the number of signs required (start of system, end of system, intermediate major interchanges, for each direction of flow-on each freeway) by an average installed sign cost. Typical sign size is on the order of 120 square feet (11.15 square meters). Normal installation is a roadside mounting. Cost estimate guidelines should be available from recent signing contracts. Sign maintenance costs should be small; they can be included however, and may be estimated from typical experience with similar types of standard roadway signs.

12.11 PRE-TRIP/ENROUTE INFORMATION SERVICES

12.11.1 Introduction

Pre-trip/enroute information services permit the motorist to optimize his route selection either before starting his trip or before entering the corridor

if he is already on the road. The motorists' responses to the information also benefit the corridor by avoiding further contribution to problem situations.

The pre-trip/enroute services can be separated into two categories: general information and real-time information. General information encompasses subjects such as location and time of recurrent traffic problems, generally preferable routes between typical locations and potential alternate routes, descriptions of IMIS, location of gas, food and lodging facilities and sources of additional information. This type of information is usually presented in the form of printed material such as brochures, newspaper columns, tourist guides and maps.

The real-time information includes current and projected traffic and roadway conditions in the corridor and can be provided in the form of recordings via dial-up telephone, radio or TV broadcasts, and announcements or scoreboard displays at recreational facilities. Real-time sources can also be used to provide other types of information as well as real-time conditions. For example, a radio broadcast could include an address from which IMIS brochures may be obtained.

12.11.2 General Information Sources

The general information sources provide the means of educating the public about the functions and potential benefits of IMIS, and therefore increase the probability of positive response to IMIS real-time outputs and acceptance of IMIS limitations. Table 38 lists the various types of printed media together with potential printing sources and location where they may be distributed. The source of original material for all of these media would be the cognizant governmental agency and the initiative for promulgating the information would rest with this agency.

Additional sources of general information other than printed media include radio and TV presentations which would be prepared in cooperation with the commercial broadcaster.

Table 38. General Information Media

Printed Medium	Printing Agency	Distribution
Brochure	State DOT	Shopping Centers, Recreation Centers, State Offices and License/ Registration Mailings
Newspapers and News Magazines	Commercial Publishers	Newsstands, Mail
Tourist Guides	State Agencies, Automobile Club and Oil Companies	Mail, Pickup at Source Agencies

12.11.3 Telephone Information

The telephone dial-up system provides motorists with current traffic conditions in the corridor by means of a short tape-recorded message. This message is updated by personnel at the IMIS control center whenever a change in conditions occurs.* All equipment necessary to perform this function is normally available from the telephone company on a lease basis.

The number of leased lines required is dependent on the expected number of calls per day and the "capacity" of each line. These are difficult to predict in advance; however, an initial estimate may be obtained from a "rule-of-thumb" developed from some limited experience. The estimate is made as follows:

$$\text{Maximum number of Calls/Day} = 2\% \text{ of Average Corridor ADT}$$

$$\text{Line Capacity} = 200 \text{ calls/day}$$

$$\text{Number of Lines Required} = \frac{\text{Number of Calls/Day}}{\text{Line Capacity}}$$

For example, if the average corridor ADT is 300,000 vehicles, the number of lines required is $.02 \times 300,000/200 = 30$ lines. This estimate is expected to be reasonable for the feasibility study. In an actual system, this could serve as a starting point and be easily modified (in either direction) as usage experience is developed.

12.11.4 Radio and TV Broadcasts

Spot announcements describing traffic problems can be helpful to motorists planning to use the corridor. Information would be made available to interested stations directly from the IMIS control center by means of a CRT terminal display with auxiliary printer. Data communication would be provided by a leased voiceband channel employing standard low-speed (300 bit per second) modems at each end. Such an arrangement would permit the station to broadcast current information whenever its schedule permitted. A blinking CRT display, audible buzzer, and/or lamp remotely controlled by the IMIS control center could alert studio personnel to unusual conditions. Past discussions with metropolitan area commercial broadcast station personnel have indicated that stations appear interested in participating in such motorist information systems. It is suggested that this feature be included in the feasibility study without verification of interest at this time since the costs involved are modest.

12.11.5 Recreational Facilities

Dial-up telephone communication between recreational facilities and the IMIS control center can be used to provide information for public address announcements or scoreboard display of traffic conditions. A call could be initiated

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*For example, such a system presently exists in New Jersey for the Garden State Parkway. Motorists dial "P-A-R-K-W-A-Y" to obtain traffic information.

by either party; however, in the event of a severe problem, it is anticipated that IMIS control center personnel would call the recreational facility. Since existing telephones can be used for this purpose, costs are considered negligible. Formal procedures between the IMIS authority and the facility operating agencies should be set up (prior to system operation) to insure that pertinent information is provided to the public whenever it is available.

12.11.6 Cost Estimates

Costs for providing the pre-trip/enroute information services of the types described in the preceding paragraphs may be determined by the using agency. Alternatively, a set of typical cost estimates are provided in Table 39.

Table 39. Typical Costs Estimates for Pre-Trip/Enroute Planning Services

Information Service	Cost Element	Estimated Cost: Capital	Maint.	Oper.
Brochure	4 pages, 1/2 million copies	$25,000*	---	---
Dial-up Telephone	leased line, each	---	---	$600/yr**
Radio/TV	IMIS Central:			
	Modems, each station	$ 1,000	$120/yr	negl.
	Computer interface	$ 5,000	$600/yr	negl.
	Computer programming	$10,000	---	---
	Radio/TV Station:***			
	Modem	$ 1,000	$120/yr	negl.
	CRT Terminal	$ 3,000	$360/yr	negl.
	Printer	$ 3,000	$360/yr	negl.
	Leased line, per station	---	---	$600/yr**
Recreational Facility	Leased line, per facility	---	---	(assumed existing)

*Local printers can be contacted for a more refined cost estimate. In-house printing capability may also be considered if available.

**More accurate value can be obtained by contacting local telephone company.

***Costs are charged to the IMIS system for the feasibility study. Actually, however, the radio stations should be willing to participate in these costs.

Because IMIS operating personnel will be required to spend some time preparing information, working with other agencies, updating tape recordings, etc., the cost of this time could be charged to the pre-trip/enroute information function. However, it can reasonably be assumed that these duties will not require adding personnel to the staff and therefore no additional cost was assumed.

12.12 EQUIPMENT CABINETS

Because of the geographic extent of an IMIS corridor and the amount of field equipment employed (particularly surveillance), a large number of cabinets is required; thus, this represents a significant cost item.

It is anticipated that two types of cabinets will be used in the system; the (larger) base mounted type, and the (smaller) pedestal mounted type. The estimated mix (based on the Long Island IMIS studies) is expected to be on the order of 20% large, 80% small.

Unit cabinet costs may be estimated from recent contracts, or alternatively the following estimates may be used:*

Large cabinet:	$1800 each
Small cabinet:	$1100 each
Average (based on mix):	$1250 each (approximately)

Associated maintenance costs are related to repair or replacement for knockdowns. Since standard traffic engineering practice is to locate cabinets to minimize the possibility of being struck by a vehicle, a large number of knockdowns is not anticipated. The user may estimate the annual number for the total corridor, or assume the following as an alternative:

10 knockdowns/year @ $1500 each

Cabinet operating costs will accrue if venting fans or heaters are used. Typical values may be estimated from past experience.

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*Since the IMIS corridor will contain hundreds of cabinets, a quantity discount of about 10 percent has been assumed in the cost estimates given.

CHAPTER 13

DEVELOPMENT OF ALTERNATIVE PRELIMINARY SYSTEM DESIGNS

13.1 INTRODUCTION

13.1.1 Objectives

- To integrate the subsystem designs and develop a series of alternative IMIS configurations for subsequent evaluations

13.1.2 Inputs

- The roadway networks developed in Chapter 9
- Relevant subsystem design data (from Chapter 12)

13.1.3 Outputs

- Corridor maps for each roadway network showing approximate locations of variable message signs and highway advisory radio areas
- The series of alternative preliminary system designs, with equipment quantities summarized in tabular form

13.2 OVERVIEW OF SYSTEM DESIGN DEVELOPMENT

The previous chapter described the basic procedures and tradeoffs for the major field based IMIS subsystems. Chapter 9 provided guidelines for configuring a set of roadway networks for the given corridor. The present chapter provides the procedures for combining these results to produce the set of IMIS candidate systems (preliminary designs). The concept used to develop these systems is based on identifying those functions and subsystems which can provide significant variations in either system cost, system benefit or both. Design experience has shown that variations in network configuration, diversion signing locations, freeway surveillance configuration (detector spacing) and control center computer/operations configuration strongly impact cost and benefits.

The design procedure is structured so that the first candidate system includes all viable corridor routes and provides all IMIS functions to the maximum extent considered desirable. This 'most versatile' system design defines the base system from which all other candidates are derived. The formulation of the remaining candidates proceeds sequentially so that each design is a progressively smaller subset of the preceding candidate.

The design proceeds sequentially along two dimensions. The first dimension is the configuration of the roadway network. The second dimension is the complement of subsystem equipment necessary to implement each system function to the desired level. Figure 17 illustrates this two dimensional concept. The design of the corridor network influences the maximum equipment levels. For example, if an arterial is eliminated when going from network 1 to 2 then the equipment complement must also decrease due to the complete elimination of CIC locations, arterial surveillance locations and diversion signing locations. This reduction occurs independent of any changes in equipment densities. With a given network configuration, design variations are possible only with reduction in equipment densities.

The specification of network configurations was considered in Chapter 9. Each network configuration is constructed as a subset of the previous configuration. The choice of the roadways to be dropped is based on a knowledge of corridor needs, the ranking of the roadways developed in Chapter 8, and judgement. In general, two to three configurations and certainly no more than four is sufficient to span the spectrum of corridor networks. Any more than four would tend to produce cost and benefit differences whose magnitudes would not be enough to make clear and concise decisions with respect to system evaluation and final selection.

The final integration of the IMIS subsystems is accomplished sequentially one subsystem at a time. The recommended IMIS design sequence for integration of the subsystems is given in Figure 18. The design integration sequence is composed of two parts. First, the field based subsystems are configured by determining applicable equipment locations and quantities (Section 13.5). The second part is the design of the communication and central control subsystems (Sections 13.6 and 13.7, respectively). The two part process recognizes the design dependency of communications and central on the number and location of the field equipment.

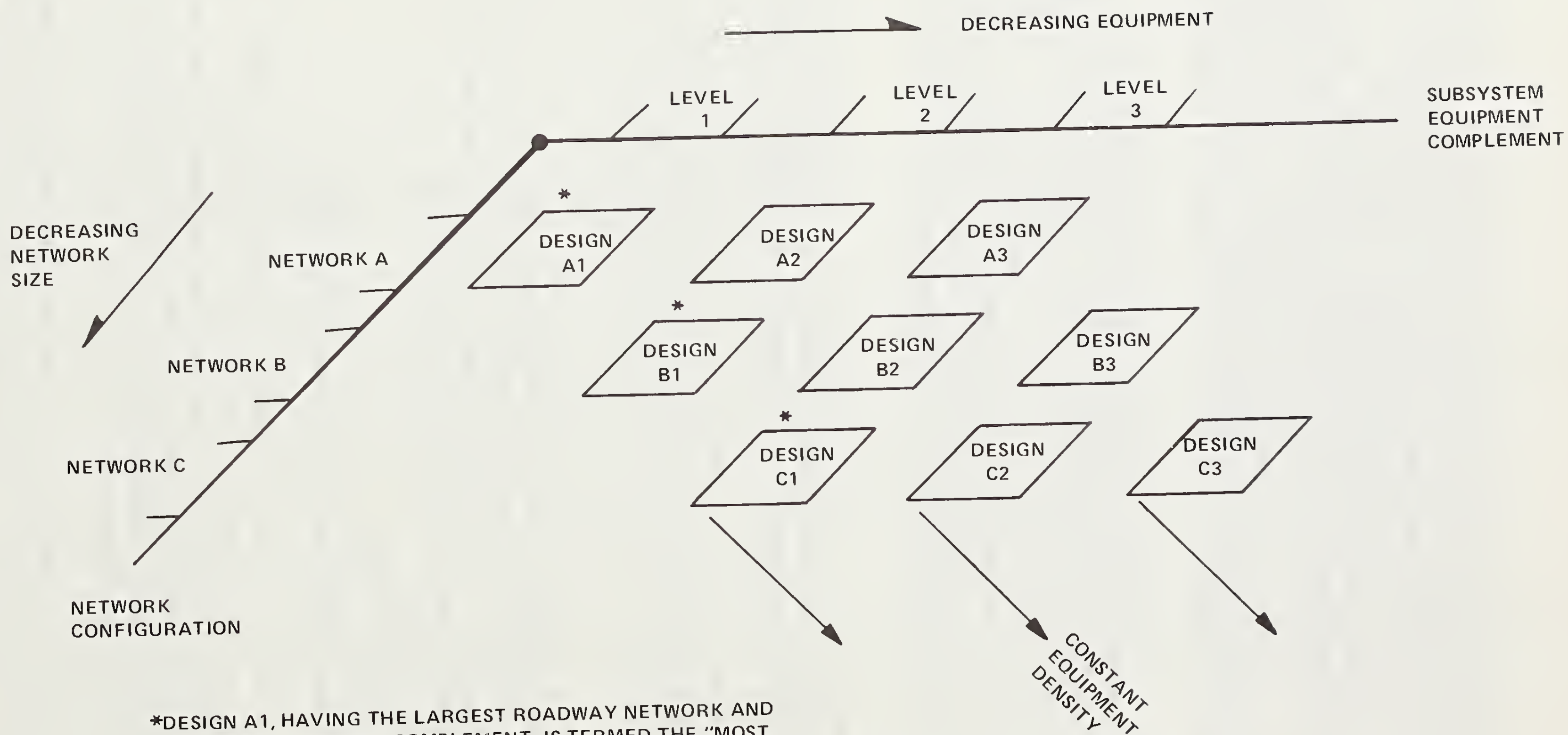
The remaining sections of this chapter provide further specific guidelines for the design of each subsystem. As each subsystem is completed, a summary sheet should be prepared for each alternate design. The summary is primarily to record quantities for later use in developing system costs. Also recall that in each case, the "most versatile" system should be configured first.

13.3 FIELD BASED SUBSYSTEMS

13.3.1 Diversion Signing Guidelines (Variable Message Signs)

For a given roadway network, candidate diversion points (and thus locations for diversion signing) are determined by considering the primary and associated alternate routes and selecting those points where a transfer of traffic can be logically made between them. Typically, diversion points will include (in order of importance) freeway-to-freeway direct access connectors, interchange connectors, freeway connectors, and arterial connectors.

Candidate diversion points for the entire corridor (network including all roadways) were identified in Section 12.4.1 as part of the procedure for determining signing configurations and unit costs. Each diversion point was assigned an



*DESIGN A1, HAVING THE LARGEST ROADWAY NETWORK AND MAXIMUM EQUIPMENT COMPLEMENT, IS TERMED THE "MOST VERSATILE" SYSTEM. THE INITIAL DESIGNS FOR EACH OTHER ROADWAY NETWORK, I.E. B1 AND C1, ARE TERMED "MAXIMUM PERFORMANCE" SYSTEMS FOR THEIR RESPECTIVE NETWORKS, SINCE THEY CONTAIN MAXIMUM EQUIPMENT COMPLEMENTS.

Figure 17. IMIS Alternative System Design Concept

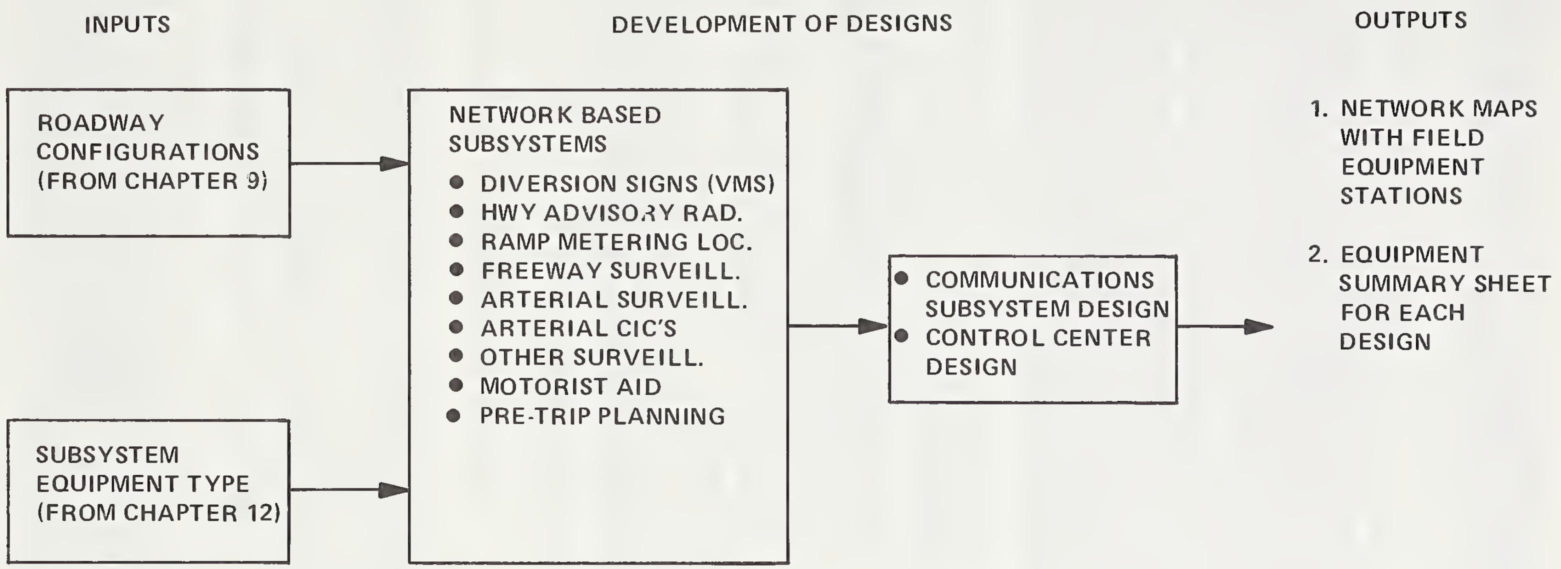


Figure 18. IMIS Design Sequence

identification number, cross-referenced to a corridor map. (These I.D. numbers should be retained throughout the study, i.e., if a diversion point is eliminated in a subsequent alternative design, the remaining diversion points should not be renumbered.)

The first system design (largest roadway network) should include all of the above diversion points. One or more additional system designs (for this roadway network) may then be obtained by (1) making selective reductions in the number of diversion points on the basis of importance, and/or (2) retaining diversion points but replacing the variable message signing with highway advisory radio.

As each new (reduced) roadway network is considered, it should first be examined to determine which diversion points are no longer applicable because alternate routes or segments have been eliminated. Once these deleted diversion points have been identified, the remaining ones represent the maximum configuration for this network. Additional system designs for the network are then determined using the same procedure as noted above, i.e., deletions and/or replacement with highway radio.

13.3.2 Highway Advisory Radio Guidelines

As an adjunct to the variable message signs, Highway Advisory Radio (HAR) stations may be located at various points in the corridor. The following uses should be considered in determining the HAR locations:

- In conjunction with variable message signs for diversion and/or traffic advisory locations.
- For traffic advisory where there are long sections of roadway between interchanges or between variable message signs.
- At the entrances to the IMIS corridor.
- At locations where there is insufficient room for a variable message sign.
- As a substitute for variable message signs for aesthetic reasons (e.g., on a parkway).
- To provide general radio coverage in the corridor.
- As a substitute for variable message signs in order to reduce system costs.

The last item should be a major consideration in developing lower cost alternatives within each network configuration, since variable message signs represent a major system cost item. Thus, some diversion points which might ordinarily be excluded to reduce costs can be retained by substituting a HAR

installation (although a somewhat reduced diversion effectiveness is to be expected). It is seen then that HAR is the only system element that may increase in going from the "most flexible" system to the lower cost alternative designs.

13.3.3 Ramp Metering Selection Guidelines

In general, metering of a particular ramp will be desirable if mainline congestion often exists in the ramp vicinity, if ramp-related accidents are frequent, or if the corridor control system will require that it be metered. The latter consideration is included primarily to cover situations where ramp volumes might significantly increase due to control measures (e.g., route diversion).

While metering of a particular ramp may be desirable, it may not always be practical. Factors which must be considered here are (1) is the ramp demand within practical metering limits, (2) is ramp storage and/or alternate diversion capability adequate, and (3) does ramp geometry permit safe metering.

Depending upon the traffic engineer's operational experience with the corridor, it may be possible to use qualitative judgment (with a minimal data base) to select which ramps should be metered. As long as the major fraction of true candidates are so identified, the result will be satisfactory for the feasibility study. Conservation can be added by retaining all "borderline" cases.

In the event that it is felt that a more rigorous study is required, Appendix F contains a set of specific guidelines which can be used to select the metered ramps.

Ramp metering has been shown to be a highly effective form of control in basic systems, and becomes even more important in IMIS because of its potential use in conjunction with other control measures. Thus, it is recommended that at least the major portion of metered ramps be retained in all alternate design configurations, unless, of course, a reduced roadway network includes elimination of a limited access facility.

13.3.4 Freeway and Arterial Surveillance Guidelines

Design of the surveillance subsystem is principally a function of the length of roadway in each network configuration. For the more extensive systems, within a given network configuration, detector station spacing on the freeways should be set at .5 miles (0.8 Km) with an increase to 1.0 mile (1.6 Km) spacing for the least extensive system designs. All entrance and exit ramps should be detectorized, whether or not metered, since ramp volumes are required for the control algorithm.

Arterial surveillance must be specified for all arterials in each network. The design of the surveillance is closely related to the level of control specified for the arterials. In general arterials selected for inclusion in a network configuration will maintain a constant level of surveillance and control over each system design. Only when an arterial is eliminated from the network will a corresponding change in arterial surveillance and control equipment be made.

As noted in Chapter 12, the number of detector stations for arterial surveillance will be either 1 per intersection or 1 per mile (per 1.6 Km), per roadway direction, for non-CIC intersections. Selection of CIC intersections was also discussed in Chapter 12, along with its surveillance requirements.

Starting with the most versatile system, a summary listing should be prepared, by roadway, indicating both the number of detector stations and the number of detectors. For the case of freeways, the mainline and ramp requirements should be tabulated separately. Then, summaries for the other alternative designs can be readily derived by eliminating the detectorization on "eliminated" roadways, or halving the requirements for any cases where the larger detector spacing is used for the freeways.

13.3.5 Other System Surveillance Guidelines

Possible other forms of surveillance for the IMIS corridor are:

- Patrolling police cars
- Patrolling tow trucks
- Closed circuit television (CCTV)

These were discussed in Chapter 12. For configuring alternative designs, all those selected should be included in the most versatile system. Other alternative designs can then be configured by reducing the quantities or eliminating the item entirely. One exception is the patrolling tow trucks, if franchising can be obtained. Since this would essentially be a no-cost item, it could be retained in all systems.

13.3.6 Motorist Aid Guidelines

The motorist aid subsystem should be configured with a call box at .5 mile (0.8 Km) intervals on both sides of the roadway along all limited access facilities. Multiplying the number of miles of freeway roadway by 4 boxes per mile (per 1.6 Km) provides the level of equipment required for each system design. If any limited access roadways are eliminated from the networks, the complement of motorist aid equipment decreases accordingly. Within a given network configuration, one variation in system design can be obtained by entirely eliminating the motorist aid subsystem. Other possibilities include instrumenting only portions of the freeway system and/or discrete freeways only. However, unless the corridor is extensive in size, this will usually not produce a significant system cost variation.

13.3.7 Pre-Trip Planning Guidelines

All of the pre-trip planning elements listed in Chapter 12 should be included in the most versatile system. They can be selectively or entirely eliminated for the lower cost alternative designs.

13.4 COMMUNICATIONS SUBSYSTEM

The objective of this section is to address the design of a communications subsystem which is capable of satisfying the data transmission requirements of IMIS, and develop a corresponding cost estimate. Communications is undoubtedly the most complex aspect of the system due to its interrelationship with all other equipment and the number of trade-off factors which can be considered. Nevertheless, in keeping with the handbook philosophy, an approach to the subject has been developed which permits the user to meet the above objective using a relatively straight-forward procedure. The methodology attempts to avoid the extensive field work and detailed trade-off studies normally required for a site specific communication subsystem design, while providing results of sufficient accuracy for a feasibility study.

Since the communication subsystem was not discussed earlier (because of its dependence on the field based subsystems), all relevant aspects are included in this section. The section begins with a brief discussion of transmission techniques and local processing aspects, and recommends the approach considered most applicable to IMIS. Next, a simple procedure is presented to estimate the number of field cabinets required and then the number of communications units. The communications medium is then discussed and a rationale developed for selecting one for use in the study. This is followed by the procedures for developing the communications subsystem cost estimates. Finally, guidelines for configuring the communications subsystem for the alternative system designs are indicated.

13.4.1 Transmission Technique and Local Processing Aspects

There are basically two methods of transmitting data between the field equipment and the central facility, i.e., direct data transfer or data multiplexing. The decision as to which is the preferable method is dictated primarily by the quantity of data to be transmitted, which directly influences the cost. As system size and number of functions increase, the trade-off becomes increasingly one-sided in favor of multiplexing. Suffice it to say that IMIS installations are large systems, and multiplexing is the obvious choice.

The next factor to consider is whether all "raw" data should in fact be sent in both directions or instead, undergo some processing at the local site. The major factor which makes the latter approach feasible is the advancement in the state-of-the-art of solid state electronic technology and the availability of low cost microcomputers with powerful computational capacity. The approach is not only feasible, but provides several advantages which tend to make it the trend for future systems. One major advantage stems from the fact that the communication facilities required in terms of channels per field unit (detector, controller, sign, etc.) vary inversely with the degree of local processing incorporated into the field equipment. Thus, a substantial reduction in communication facilities, which represent a major system cost item, is possible. Local processing also influences the communications technique employed. In general, for systems that do not require local processing, Frequency Division Multiplexing (FDM) is often more cost-effective, while with local processing, Time Division Multiplexing (TDM) using serial data transmission with multipoint polling is more cost-effective.

In the communications trade-off study performed for the Long Island, N.Y., IMIS, it was found that a 50-pair (average) cable and 400 TDM units could handle the total system data transmission requirements if local processing was used. Without local processing, a 300-pair cable and 3,000 FDM units (transmitter/receiver pairs) would be required at more than double the cable/equipment cost. The former case resulted in a savings for the communications subsystem costs of greater than 20 percent. It is also noted that in this case, the communications subsystem represented the largest single cost element, comprising about 40 percent of the total system capital costs.

Other advantages of local processing include reduced computational burden on the central facility computer, simplification of interfaces through microprocessor programming flexibility (e.g. can use standard modems*), more sophisticated back-up modes on a local level, more advanced fault diagnostic capability, and generally greater operational flexibility.

Based on the foregoing, it is recommended that the local processing approach (with TDM) be adopted for the feasibility study. All required local data processing, formatting, and interface functions are then performed by the "communications unit", which in effect consists of a microcomputer plus a standard modem.

Should the user for any reason choose an approach other than the local processing approach, the estimating procedures contained in this section can still be used, and a subsequent adjustment (increase) then applied to the final cost estimate for the communications subsystem with the local processing approach.

13.4.2 Estimate of Number of Field Cabinets Required

Various elements of field equipment including such items as detector electronics (mainline, ramp, arterial), variable message sign controllers, ramp controllers, and communications units, will be housed in roadside cabinets.**

Development of a precise cabinet number estimate is a detailed and time consuming process, since it involves determining and laying out the specific desired locations of all field equipment, determining which equipment can share a cabinet, considering adjustments in equipment locations to minimize both construction costs and the number of cabinets, and substantial field verification of the suitability of all equipment and cabinet locations. This type of effort is ordinarily left for the final system design/PS&E stage.

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*Modems, or modulator-demodulator sets, convert data to the required form for transmission (modulation) and, at the receiving end, re-convert the data to the original form (demodulation).

**Motorist aid phone enclosures and roadside radio transmitter cabinets are treated separately under their respective subjects.

For the purposes of a Feasibility Study, a reasonable estimate of the required number of field cabinets may be obtained using the following procedure:

A. For each limited access facility, use the relation:

$$NC_F = NFDS + NFR \tag{7}$$

where: NC_F = estimated number of cabinets required on the freeway

NFDS = number of freeway detector stations (from paragraph 13.3.4)

NFR = number of freeway ramps, including both entrance and exit ramps (from paragraph 13.3.4).

The estimating procedure is considered to be accurate to 10 percent or better, since the freeway detector stations and ramps (ramp detectors, and ramp controllers where metering is used) represent, by far, the largest number of equipment stations. Other equipment, such as variable message sign controllers, can normally be accommodated in these cabinets.

B For each arterial:

If average spacing between signalized intersections is equal to or less than one mile (1.6 km) the following relation should be used to determine the number of cabinets:

$$NC_A = NSI, \tag{8}$$

or if the average spacing between signalized intersections is greater than one mile (1.6 km):

$$NC_A = TMS \tag{9}$$

where: NC_A = Estimated number of cabinets required on the arterial.

NSI = Total number of signalized intersections on the arterial

TMA = Total number of roadway miles of the arterial.

As in the freeway case, the detector requirements serve as the determining factor for cabinet locations. The above two situations are stipulated to provide either one cabinet per intersection (the minimum requirement), or additional cabinets for intermediate detector stations when signalized intersections spacing is large. The requirements for and locations of the additional detectors and cabinets would be determined during final design as a function of specific traffic and geometric conditions (e.g., presence of non-signalized intersections, local traffic generators, known bottleneck locations, etc.).

Where the characteristics on different sections of a given arterial vary substantially as regards signal spacing, each may be treated separately and the results summed to yield the total number of required cabinets.

The foregoing assumes that existing controller cabinets at signalized intersections are not used for the additional equipment, except possibly for a small interface unit. It is recommended that this assumption be retained for the feasibility study unless replacement of existing cabinets is planned before system implementation.

C. Total Number of Cabinets:

The total number of system field cabinets required (NC_T) is simply the sum of the number of freeway and arterial cabinets.

13.4.3 Estimate of Number of Communications Units Required

It is necessary to connect each field equipment cabinet to the central facility via the communications medium to enable the required transmissions between the two. However, it is not always necessary to provide a communications unit in each cabinet, since DC signals can be brought from detectors and other equipment in one or more cabinets, via direct connection, to a cabinet which contains a communications unit. The purpose of such an approach is, of course, to reduce costs.

With local (field) data processing, the processing is performed by the communications unit, which, as noted earlier, consists of a microcomputer plus a modem for data formatting. The cost of such a unit is on the order of $2000. The major trade-off factor, then, is the cost of the additional wire pairs (to interconnect the cabinets) versus the cost of the communications units.

To provide an indication of the trade-off, consider the case of cabinets spaced 1/2 mile (0.8 km) apart. Let one cabinet contain a communications unit, and assume that this unit serves the nearest upstream and downstream cabinets. Assume further that about 5 cable pairs would be required (primarily for detectors) to interconnect the cabinets. The cost of the additional wire is approximately $10.00/pair/1000 ft. ($32.80/pair/1000 meters). Thus the cabinet interconnect cost is about

$$5 \times \$10 \times 5.280 = \$264$$

The above indicates the desirability of the approach of not placing communications units in each cabinet, but rather designating only certain cabinets as "telemetry cabinets," i.e., those which will include a communications unit.

The next question is how many cabinets should be connected to each "telemetry cabinet"? A more rigorous trade-off study could be performed to establish this. However, as the number increases, other factors such as increased complexity of the communication unit and reliability (loss of one unit results in loss of larger amounts of data) begin to become important. For the purposes of the feasibility study, it is suggested that a relatively conservative approach be taken, that is, to assume as in the example given, that each telemetry cabinet services its 2 adjacent cabinets. Thus, the number of communications units required for each roadway, NCU, is simply one third of the total number of cabinets computed in paragraph 13.4.2. The total for the system is obtained by summing the number of units determined for each roadway.

13.4.4 Communications Medium Selection

For the present state-of-the-art, three different media could be used to satisfy the communications requirements for IMIS, i.e., owned cables, leased telephone channels, and radio. These media can be used separately or in combination as hybrid subsystems. The factors to be considered in selecting a communications medium include reliability, maintainability, flexibility, and cost (initial and operational), as described in the following paragraphs.

Reliability of the medium and its associated equipment is a function of the design and quality of the equipment used, the care with which it is installed and checked out, and the degree of susceptibility to damage caused by construction work, vandalism, or natural phenomena (wind, flooding, freezing, etc.). Underground cable (coaxial or wire pair) has potentially the highest reliability of the candidate media, provided that high quality cable is used, and care is taken in handling and splicing. Coaxial cable is more vulnerable to mishandling than wire-pair cable, and its required line amplifiers are susceptible to water damage unless placed above ground (where they become more susceptible to accidental damage or vandalism). Leased channel reliability is unpredictable because it is dependent on telephone company activities, which can cause outages or electrical noise on the lines. Reliability of radio transmission can be affected by atmospheric conditions, changes in transmission paths due to construction or tree growth, and vandalism of the required tall antennas located at each cabinet.

Maintainability of the communications equipment and transmission medium is a function of accessibility, equipment modularity, and the personnel skills required to diagnose faults and service the equipment. If contract maintenance is employed, these factors will be reflected in the cost of the service. The differences in maintainability among the techniques considered are primarily in the personnel skills needed, with coaxial cable and radio requiring the highest skill levels. Leased channels, of course, are maintained by the telephone company, which may result in longer outages (lower reliability) than if servicing is performed by an independent contracted organization or by own forces.

Flexibility for expansion or change can be provided by any of the techniques described. Radio is probably the most flexible, provided that spare channel capacity has been requested and granted by the FCC. Additional leased channels may also be available at most locations, but if new facilities must be built, delays in implementation and added construction costs could result. The owned cable approaches normally provide spare capacity, but expansion beyond the areas where initially installed would require additional construction effort.

The cost of coaxial cable installation is approximately the same as for wire pair cable (with the lower cost of coax offset by the added care required in cable handling), and the cost of two-way communications equipment is about the same. However, a wire pair cable must also be installed with the coax for

cabinet interconnection.* An additional cost results from the need for line amplifiers every 2000 to 4000 feet (610 to 1220 meters). Cost of equipment installation, check-out, and maintenance will be higher for coax than wire pairs because of its greater complexity and the higher skill personnel required. Overall, then, the total cost of coax (initial plus operating) is usually higher than for wire-pair cable, typically by about 10 to 20 percent.

Radio equipment that meets FCC requirements for communication in the 952 to 960 MHz region of the microwave (UHF) spectrum is currently available. Six voiceband channels may be multiplexed on each assigned 100 KHz frequency band of the 32 bands available. A radio communications system would typically consist of a transmitter/receiver and antenna at each cabinet site, plus a number of "backbone" units mounted on high ground to serve as repeaters for the more distant units. The cabinet equipment consists of several book-size plug-in modules, and the antenna typically consists of an open grid truncated parabola about 4 feet (1.2 meters) wide mounted on a pole tall enough to clear tree tops. Equipment cost is about $6,000 per site, plus additional costs which result from the need to inspect each site (to ascertain adequacy of transmission path) to check out the system, and to calibrate each transmitter at least once each year. Maintenance costs will be high because of the high-skill personnel required. It is difficult to make a relative comparison of total costs for radio communications versus cable because of site-specific factors. However, in a recent IMIS study for the northern Long Island Corridor in New York, the total equivalent capital costs for radio communications (initial plus present worth of recurring costs) was slightly lower than owned cable pairs (by about 8 percent).

Costs for leased lines are the most difficult of all to generalize, and in fact, cannot be determined to any reasonable degree of accuracy without consultation with the local telephone company or companies involved.** Factors affecting costs for a leased line system include such items as availability of spare channels in the vicinity, distances from existing telephone service to the field equipment cabinets, number of drops required, amount of new construction required, number of central offices and jurisdictions involved, and, of course, the local tariff structure. The problem of generalizing such costs can perhaps best be illustrated from the following two case histories.

A communications trade-off study was performed for the "Single Point Diversion System Project" in the Baltimore, Md., area, considering owned cable versus leased lines. For the two roadways involved (I-695 Baltimore Beltway and the Harbor Tunnel Thruway), utility service poles containing both power and telephone lines were located very close to the roadways (generally within 100 feet (30.5 meters)). Furthermore, spare channels were available at virtually every site, thus obviating the need for any significant construction. This, coupled with

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*The alternative would be to put a communications unit in every cabinet; however this is a more expensive approach.

**A further problem is the uncertainty of future tariff increases. Recent experience indicates doubling and tripling in some cases, which are claimed necessary by the telephone companies to adjust for previous inordinately low tariffs used for such service.

the existing tariff structure (airline miles type) resulted in the leased line system cost being only a fraction of the equivalent cost of owned cable. A similar trade-off study was conducted for the IMIS Corridor in northern Long Island, N.Y. In this case, existing spare channel capacity was generally unavailable along the freeways, and the effort to make a determination of where and how much was available at specific locations was claimed (by the Telephone company) to require an inordinate amount of engineering effort. Thus, the "leased" system to be provided by the telephone company would have consisted of the telephone company installing their own cable throughout virtually the entire freeway system, with multipoint drops, plus connections to local utility poles along the arterials. The resulting total equivalent cost estimate was roughly equivalent to that of the user owned cable case.

In the former (Single Point) case, the cost difference was so great as to override the normally preferred owned cable, and the leased system was recommended. In the latter (IMIS) case, the owned cable was easily justified.

As a result of such past experiences, the user is cautioned against attempting to develop costs for leased line systems "on his own", and advised (if interest exists in a leased system) to develop cost estimates in conjunction with the local telephone company. They should provide inputs as to the cost of new construction which they require, and they must provide their applicable tariff structure to enable computation of recurring costs. An example of the latter (recurring costs) from the IMIS study is shown in Table 40. Note that in this example, three different counties (Suffolk, Nassau, Queens), and thus three telephone jurisdictions were involved. Also, three different cases were considered to show the effect of reducing the number of leased channels required, by interconnecting cabinets. The 850 cabinet and 600 cabinet results represented two different candidate system being considered in the trade-off study.

The foregoing discussions of relative costs of owned cable, radio, and leased lines have been based on comparisons of "total equivalent capital costs" i.e., the sum of the capital costs (initial) plus the present worth of the annual operating (recurring) costs. Regardless of the method used to combine the costs (for example, capital costs could be "annualized" and added to the annual recurring costs), the fact remains that both must be taken into account in a trade-off study and in the eventual system evaluation. In the benefit/cost ratio the components of initial and recurring costs lose their individual identities as a result of the need to properly account for the "time value of money". The individual components, however, are of substantial importance to the operating agency, due to the fact that there is Federal Government cost sharing for initial costs but none (at present) for recurring cost. Thus, the recurring costs must at least be considered, perhaps as a constraint, to avoid development of a system design which the responsible agency cannot afford to operate year after year, despite the benefits accruing to the motoring public.

As an example of the relative range in recurring costs associated with the different communications media, the following results from the Long Island Corridor

Table 40. Example of Leased Channel Cost Estimate

ID	COST FACTORS	AMOUNTS	
	Case 1: All cabinets on leased channels		
A	Allowable No. of loops/multipoint channel	13	
B	Loop cost	15.28 $/Mo.	
C	Allowable No. of data bridges/multipoint	3	
D	Data bridge cost	89 $/Mo.	
E	(A + 1) x B + C x D (see footnote*)	481 $/Mo.	
F	Suffolk trunk distance	9.8 Mi	
G	Suffolk avg. bridge to C.O. distance	1 Mi	
H	Suffolk mileage cost	9.28 $/Mi/Mo	
I	Fraction of cabinets in Suffolk	.25	
J	(E + F x H + G x H x A) x I/A	13.2 $/Mo.	
K	Nassau trunk distance	25 Mi	
L	Nassau avg. bridge to C.O. distance	4 Mi	
M	Nassau mileage cost	3.94 $/Mi/Mo	
N	Fraction of cabinets in Nassau	.45	
O	(E + K x M + L x M x A) x N/A	27.2 $/Mo	
P	Queens trunk distance	34 Mi	
Q	Queens avg. bridge to C.O. distance	3 Mi	
R	Queens mileage cost	2.17 $/Mi/Mo.	
S	Fraction of cabinets in Queens	.30	
T	(E + P x R + Q x R x A) x S/A	14.8 $/Mo.	
U	J+O+T (leasing cost per cabinet for data transmission)	55.2 $/Mo.	
V	Maintenance jack leasing cost	2 $/Mo	
W	Motorist Aid phone leasing cost	2.85 $/Mo	
X	U+V+W (total leasing cost per cabinet)	60 $/Mo	
Y	Present worth (15 yrs. @10%) plus one time install. charge of $0.3K	5.8 K$/Cabinet	
	Total Cost, 850 Cabinets		4.9 M$
	Total Cost, 600 Cabinets		3.5 M$
	Case 2: One-half cabinets on leased lines; one-half DC-connected		
AA	Extension cost	3.94 $/Mo	
BB	(X+AA)/2 (leasing cost)	32 $/Mo	
CC	Present worth (15 yr.@10%) plus $0.3K	2.9 K$/Cabinet	
	Total Cost, 850 Cabinets		2.5 M$
	Total Cost, 600 Cabinets		1.7 M$
	Case 3: One-third leased; two-thirds DC-connected		
DD	Extension Cost	7.88 $/Mo	
EE	(X+DD)/3 (leasing cost)	23 $/Mo	
FF	Present worth (15 yr.@10%) plus $0.3K	2.1 K$/Cabinet	
	Total Cost, 850 Cabinets		1.8 M$
	Total Cost, 600 Cabinets		1.3 M$

(Note: 1 mile = 1.609 kilometers)

*The term A+1 includes "A" field cabinets + 1 central (total of 14 points).
K$ = Thousanads of dollars; M$ = Millions of dollars

Source: Long Island IMIS Feasibility Study

IMIS Study are noted below:

• owned cable pairs	$44,000/year
• owned coaxial cable	$100,000/year
• radio	$240,000/year
• leased telephone lines	$381,000/year

(The leased line value would, in addition, be expected to "inflate" much faster than the others). The "total equivalent costs" (capital plus equivalent recurring) were, however, quite close. The owned cable pair medium was finally selected because it is a proven technique, permits complete control by the operating agency, provides good reliability, maintainability and flexibility, and has the lowest operational cost. Had closed circuit television been included in the system, the coaxial cable would have been the choice for reasons of higher bandwidth requirements.

No attempt to prejudge the selection of a communication medium for a given system is intended by the foregoing discussion; rather it is intended to highlight the major factors to be considered. Other site-specific items, such as availability of existing communications facilities with spare capacity, should also be taken into account. Cost is undoubtedly the major factor; however, due consideration must be given to the using agency's ability to operate, control, and maintain the communications system.

As noted previously, the major trade-off factor in the communications subsystem is the medium cost. Normally, a fairly substantial effort is involved to develop reasonably accurate costs for each candidate to be considered. In an attempt to reduce this effort, a procedure is recommended which entails development of cost estimates for one medium, and subsequent cost adjustment for others (except leased lines*) if they are being considered as serious candidates. The particular medium recommended for cost estimating is owned cable pairs, for the following reasons:

- It is expected to be the most commonly used medium if CCTV is not a requirement. If CCTV is a requirement, it is most likely that coaxial cable (rather than radio) will be used, and the major construction cost factors are similar to those for cable pairs.

- The estimating procedure involves cost elements which are most familiar to traffic engineers (e.g. trenching, jacking under pavement, hanging conduit, etc.), and for which good cost estimates are readily accessible either from recent contracts or through contacting local construction firms.

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*Adjustment factors cannot be developed for a general application because of site-specific dependence and potential wide variance as discussed previously.

- Even if no cost adjustment factors were applied, the costs for an owned wire pair cable medium is expected to be close enough to the other candidates so as to have no significant effect on the feasibility study outcome. (Again, leased lines are the exception, and must be treated separately).

Of course, the final choice of how detailed a study can or will be performed rests with the agency performing the study. The approach presented herein is offered as one such alternative.

13.4.5 Communications Subsystem Cost Estimation

The following paragraphs present the procedure for developing the communications subsystem cost estimate. The estimates for the medium (cable) are considered first, followed by those for the communications equipment. In each case, cost are broken down into the capital, maintenance, and operations categories. The medium and equipment costs in each category are then totalled to yield the communications subsystem costs. Finally, factors are presented to adjust the subsystem costs if an alternative medium is being considered.

In the development of medium cost, it is convenient to concurrently include the construction cost for provision of AC power, since the procedure is similar and in fact may involve some common trenching.

A. Estimate of Number of Cable Pairs Required

There are 2 basic options for installing cable on freeways. One is to use cable on both sides of the roadway; the other is to use cable on only one side of the roadway with crossovers to pick up equipment on the other side. The basic tradeoff here is one of cost versus reliability. The dual cable approach is generally more costly (typically by about 15 percent, but quite site-specific depending mainly on the available crossover spacings). It is also obviously more reliable, and easier to maintain because of its lesser complexity. For the feasibility study, it is recommended that the dual cable configuration be used, since its cost will be representative and generally conservative.

For arterials, the reverse situation generally applies, that is, a cable along only one side of the arterial is normally used, due to the substantially higher cost of cable installation in this case. Therefore, a single cable configuration is recommended here for use in the cost estimating.

An accurate determination of the number of cable pairs required is an involved process, requiring specific information on the quantities of each item of field equipment, polling rates, word lengths required for transmission of each type of information, the number of equipment cabinets, and the number of cabinets containing communication units ("telemetry" cabinets). Such specific information is usually not developed until the final design/PS&E stage.

For the purposes of the feasibility study, an estimating procedure has been developed which is simple to apply, yet sufficiently accurate for the intended purpose. The procedure is based on the fact that the electronic surveillance (detectors) imposes the major communications requirements. Thus, estimates made using these quantities, plus a subsequent adjustment for the other equipment elements, will provide a reasonably good measure of cable pairs needed.

The procedure consists of the following steps which are performed for each roadway:**

(1) List the number of detectors, ND, (from paragraph 13.3.4)

(2) List the total number of field communications units, NCU, (from paragraph 13.4.3)

(3) Calculate the required data rate, DR, as follows:*

$$DR = \left[\frac{ND \times NB_D + NCU \times NB_A}{PP}\right] \times 1.25 \text{ (in bits/seconds)} \qquad (10)$$

where:

NB_A = serial message length (number of bits) required for detector data transmission

NB_A = serial message length (number of bits) required for cabinet address and check bits

PP = polling period, in seconds

1.25 = an adjustment factor for accommodating all other equipment requirements.

Typical values for NB_D, NB_A, and PP are 30 bits, 30 bits, and 30 seconds respectively, for the local processing, TDM polling approach.

Example: *Assume there are 800 detectors and 100 communications units. The required data rate would then be:*

$$DR = \left[\frac{800 \times 30 + 100 \times 30}{30}\right] \times 1.25 = 1{,}125 \text{ bits/second}$$

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*The computation assumes separate wire pairs for each direction of transmission.

**Note that for freeways, only one flow direction is considered in the calculation to establish cable size. The other flow direction (i.e., the other side of the road) will then also have a cable of the same size.

(4) Calculate the minimum number of cable pairs required for data handling, NP_D, as follows:

$$NP_D = \frac{DR}{RPP} \tag{11}$$

where RPP is the assigned rate per pair. A conservative estimate for RPP is 600 bits/sec.

(5) Calculate the minimum number of pairs NP_L, required to avoid excessive line loading, as follows:

$$NP_L = NCU/20. \tag{12}$$

(6) Select either NP_D, or NP_L, whichever is greater.

Using the previous example, the minimum number of pairs required for data handling would be:

$NP_D = \frac{1,125}{600} = 2$ *cable pairs, and the minimum number required to avoid loading would be:*

$NP_L = 100/20 = 5$

Thus, five pairs would be required.

(7) Add additional cable pairs, as follows:

- 5 pairs for direct connection of equipment field cabinets to the "telemetry" cabinets (i.e., field cabinets which also contain communications units).
- 1 pair for each highway advisory radio (HAR) site
- 1 pair for each 20 motorist aid phones

(8) Sum the number of pairs obtained in steps 6 and 7. Add at least 20 percent for spares (minimum of 2 pairs), and the resulting total will determine the minimum number of pairs required.

(9) Select a "standard" cable size, i.e., the smallest one which provides at least the number of pairs required. For example, if 22 pairs were required, select at least a 25 pair cable; if 42 pairs were required, select at least a 50 pair cable, etc. Typically, standard cables are available with the following numbers of pairs: 6, 12, 18, 25, 37, 50, 75, 100, etc.

It should be noted that the actual cable size would not be constant over its length, but rather, would start with a small cable at the point furthest from the central facility and increase in steps toward the central facility. The cable size calculated above is a representative average and is used only for estimating cable costs.

B. Cable Cost Estimate

A worksheet, as typified by Table 41, may be used to develop the cable cost estimate. The table identifies the inputs required and suggested measurement and cost units.

In general, the unit costs used should be based on either recent contract experience or discussions with local contractors. It is normally desirable to conduct a field survey to estimate the amount or percentage of roadway or roadway border surface which falls into each trenching or plowing category. This can be accomplished by driving each roadway once during off-peak periods, making coded notations on a map. Items such as numbers of bridges and ramps and length of mainline cable can be obtained from maps and plans.

Since specific equipment locations are not defined in the feasibility study, certain distances (lengths) are specified as averages, as noted in the table. The same applies to such items as ramp widths and bridge (underpass or overpass) lengths to obviate the need for many individual measurements. Typical average lengths can be estimated during the field survey and/or using maps and plans.

The number of freeway splice boxes required (item M in the table) is a function of the length of the cable reel used, the number of cabinets available, and the number of transitions from direct burial to conduit (e.g., for jacking under ramps or hanging on bridge structures). It is generally recommended to bring the cable into a cabinet terminal board when a cabinet is available, instead of installing an adjacent splice box. Cable lengths of 2500 ft. (762 meters) are available, and this normally exceeds the cabinet-to-cabinet spacing. Therefore, intermediate splice boxes should not be required. For cable transitions across ramps and bridges (underpass or overpass), normally one splice box is required on each side. However, there will usually be a cabinet located in these areas, so that fewer splice boxes will be required. A reasonable estimate for the number of splice boxes may be obtained as follows:

$$\text{No. of Splice Boxes} = \left[\text{No. of ramps} + \text{No. of bridges}\right] \times 1.5 \qquad (13)$$

where the factor 1.5 represent the average condition between having one cabinet at each location (therefore needing only one splice box) and having no cabinets at certain locations (e.g. at an interchange with several ramps, only one of which has a nearby cabinet). In the case of arterials, the trenching/conduit/cable installation cost used should include splice boxes.

The number of freeway cabinets can be obtained from the estimating procedure given in paragraph 13.4.2.

Table 41. Typical Cable Cost Estimate Worksheet

ID		Unit	QTY	Cost (M$)
A	Freeway earth border length	Kft.		
B	Cable plowing and splicing cost	$/ft		
	A x B x 1000			
C	Freeway hard or steep border length	Kft		
D	Trenching or mounting conduit	$/ft		
	C x D x 1000			
E	No. of bridges	No.		
F	Avg. distance along fwy	Ft.		
G	Installed Conduit Cost	$/ft		
	E x F x G			
H	No. of ramps	No.		
I	Avg. width of ramp	Ft.		
J	Jacking or Trenching Cost	$/ft		
	H x I x J			
K	Length of cable	Kft		
L	Cable cost	$/Kft		
	K x L			
M	No. of splice boxes	No.		
N	Cost of splice box (installed)	$		
	M x N			
O	No. of freeway cabinets	No.		
P	Avg. distance to main cable	ft.		
Q	Plowing/Trenching cost	$/ft		
	O x P x Q			
R	No. of freeway AC power sources	No.		
S	Avg. distance to main cable	ft.		
T	Plowing/Trenching cost	$/ft		
	R x S x T			
U	Avg. AC plow distance/cabinet	ft		
V	Incremental cost of common plowing	$/ft		
	O x U x V			
	SUBTOTAL (Freeways)			
AA	Arterial Hard shoulder length	Kft		
BB	Trenching/conduit/cable cost	$/ft		
	AA x BB x 1000			
CC	Arterial Soft shoulder length	Kft		
DD	Trenching/conduit/cable cost	$/ft		
	CC x DD x 1000			
EE	Existing (or planned) conduit length	Kft		
FF	Cable replacement	$/ft		
	EE x FF x 1000			
GG	AC Power Cost	$		
	SUBTOTAL (Arterials)			
	TOTAL			

(Note: 1 ft = 0.3048 meters,
$1/ft = $3.28/meter)

Regarding the provision of AC power for freeway field equipment, a typical configuration (if permitted by code) is to bring the source power to the nearest point of the cable trench, and then run in the cable trench (usually with vertical separation from the signal cable) to the cabinet.

Alternately, the power source can be run directly to the cabinet through a separate trench. If utility drawings (secondary power) are available for the corridor, these may be used to estimate distances. It is recommended, however, that the general configuration be discussed with the utility company, and their assistance be obtained in determining the power distribution system and associated construction costs. (Any new construction costs charged should be included). Power requirements are nominally small for system equipment at each cabinet, particularly since most contain only detectors and communications units. The following listings provides some typical values for various equipment elements:

Detectors (each)	10 watts
Communications Units (each)	20 watts
Variable Message Sign (Night-time Illum. for Disc Type) (each)	1000 watts
Motorist Aid Telephone (Internal and External Illum) (each)	100 watts
Ramp Metering Controller (each)	20 watts
Ramp Metering Signal Heads (2)	135 watts

It is usual to include a duplex convenience outlet in each cabinet (on a separate circuit breaker); thus appropriate wire size to handle expected outlet power requirements should be included.

For arterials, power is available at signalized intersections, and may be available at intermediate locations where intersections are widely spaced. Again, the utility company should be consulted to determine general availability and any potential construction costs to be charged if new service must be added. A similar approach may be used as for the freeway case, i.e. cable trench to cabinet or directly to cabinet, depending on number of sources available, distance, and code requirements. A few typical cases which are representative of the types of situations encountered should be considered (rather than each location individually) and typical costs or incremental costs developed for these conditions. These can then each be multiplied by the number of locations (cabinets) in the category, and a total cost developed by summing the individual category totals. The value can then be added to the arterial cost calculation on the worksheet.

Finally, if there is existing usable (or planned) conduit available of any significant length, this length may be deducted from the system total and treated instead as an incremental cost for cable replacement (replace existing cable and add new cable).

The foregoing has discussed the capital costs for the communication medium (including AC power). There are no operating costs, per se, for cable; there are however maintenance costs. As noted earlier, this will be a function of the cable quality and workmanship and handling care during installation. There is, in addition, the possibility that the cable could be damaged or cut when other construction work is performed in the vicinity. Because of these variables, it is difficult to provide a universal estimate which can be applied to any system or location. If the using agency has had or knows of cable maintenance experience, this could serve as a guide. In the absence of such data, an annual cost estimate of between 0.5 and 1.0 percent of the capital cost may be used for the feasibility study.

C. Communications Equipment Cost Estimate

The total cost for the communications units may be determined by multiplying the number of communication units, NCU by the unit cost for the equipment. The number of units is estimated as described in paragraph 13.4.3. Average unit costs may be obtained from equipment manufacturers or suppliers. Typically, the costs run about $2,000 each, which includes installation in the cabinet and checkout. (Cost of the cabinet itself is treated separately elsewhere).

Maintenance costs may be estimated as one service call per year per unit, times an average rate per call. A value of $150 per call may be used for the latter in the absence of more definitive information.

Operating costs (electricity) for the communications units are usually small, however, they can be computed as follows:

$$\begin{aligned} \text{Annual Cost} &= \text{Number of Units} \times \text{Power required per Unit (Watts)} \\ &\times \frac{1}{1000} \frac{\text{(Kilowatts)}}{\text{Watt}} \\ &\times 8760 \ \frac{\text{(hours)}}{\text{year}} \times \text{Power cost (dollars/kilowatt hour)} \end{aligned} \tag{14}$$

It is noted that power costs for other than communications equipment are not included in the communications subsystem cost, but rather with their individual operating costs. Only the (capital) cost of the power cabling is included with communications (for convenience).

D. Communications Subsystem Cost Summary

A short summary table should be prepared for this subsystem for later use as an input to the overall system costs. The table can be of the form shown below.

Item	Capital	Maintenance	Operating
Cable Installation Cost (Incl. Power)	( )		
Communications Units	( )		
Cable Maintenance		( )	
Comm. Unit Maintenance		( )	
Total Operating			( )
Totals	( )	( )	( )

13.4.6 Adjustments for Other Media

The above costs for the owned wire-pair cable can be adjusted to obtain an estimate for other communications media using the following relationships:

- Owned Coaxial Cable

  Total Capital Cost = 1.1 x Total Wire-Pair Capital Cost
  Total Maintenance Cost = 2.3 x Total Wire-Pair Maintenance Cost
  Total Operating Cost = 1.2 x Total Wire-Pair Operating Cost

- Radio

  Total Capital Cost = 0.8 x Total Wire-Pair Capital Cost
  Total Maintenance Cost = 5.5 x Total Wire-Pair Maintenance Cost
  Total Operating Cost = 5 x Total wire-Pair Operating Cost

- Leased Line System

  As noted earlier, there is too great a variation in leased line costs from one location to another to permit a generalization of these costs. The only way to develop a reasonably valid estimate is to contact the local telephone company

13.4.7 Communication Subsystem Design Guidelines

Variations from the most versatile system are essentially defined by the specific alternative designs. Where roadway networks are reduced, the communications facilities for the excluded roadways are eliminated. Where freeway detector spacing is increased, there is a corresponding reduction in the number of cabinets and communications units. Cable size should be maintained as a constant even for the smallest systems, to provide a future growth capability. Other subsystem elements do not substantially influence communications facilities; thus, as they are varied, it may be assumed that there is no corresponding change in the communications requirements.

13.5 CONTROL CENTER

The IMIS control center will house the operating staff and all equipment required to manage traffic within the corridor. The central computer will accomplish all data processing functions and will provide automatic monitoring and control of field equipment. It will also present information to the operator(s) by means of a dynamic map display, a video (CRT) terminal, a keyboard/printer (Teletype) terminal, and a line printer. Keyboards and a control panel will provide operator interface to the computer. The communications facilities will include digital data modems and their associated computer interfaces for transferring data between the control center and the roadside field equipment.

In addition, telephone facilities will permit operator contact via leased lines with the police offices, commercial radio stations, and large public gathering places, such as sports stadiums. A tape recording unit will be connected to the roadside radio playback units for updating audio sign messages, and a motorist information recording unit will be connected to the telephone dial-up network. (The cost of these two units is included in their respective subsystem costs.)

In the following paragraphs, the equipment and staffing requirements for the control center are discussed. Typical costs are then provided.

13.5.1 Equipment Configuration

The recommended design of the computer configuration for IMIS is based on recent and continuing increases in minicomputer processing capabilities and reduction in prices of both minicomputers and microcomputers. These factors have resulted in recommendation of a computer/communications design employing microcomputers in each roadside (field) cabinet designated as a communication point, and a high performance minicomputer at the control center.

The field microcomputers perform computation and data storage functions that permit use of relatively economical low speed communication techniques. These functions include communications data handling as well as processing of data associated with vehicle detectors, signal controllers, and variable message signs. This processing also reduces the central processing load, permitting selection of a central computer from among a range of commercially available minicomputers.

The data processing functions that determine the characteristics of the central computers are as follows:

- Communications control (formatting, synchronization, error checking, etc.)
- Processing system surveillance and equipment monitor data
- Processing the control algorithm

- Transmission of control algorithm output data, including sign message selection, traffic signal timing patterns and ramp metering information

- Organizing and outputting data for map, terminals, and printer

- Receiving and responding to operator input commands

- Detection of equipment failures

The processing speed and memory capacity required to perform these functions may be estimated by first establishing the rates at which the functions must be performed and the number of elements to be handled (communication lines, detectors, ramps, intersections, signs, etc.) Then the measured computer processing time and memory required to perform corresponding functions in an existing system (e.g. UTCS), together with the processing time and memory required to run the control algorithm are used to estimate the type of computer needed to handle typical corridors of different sizes.

This type of analysis was performed in a recent study.* The results showed that for an IMIS corridor with no field processing, a single minicomputer would have no spare processing-time capacity, and would require approximately 250,000 words of main memory using memory overlay techniques. With the high degree of field processing being recommended for IMIS, processing time requirements for the surveillance and interface functions (60 percent of total time required) will be reduced by at least 50 percent, resulting in at least 30 percent spare processing-time capacity. The memory requirements would also be reduced, but not by a significant amount. Under these conditions, any one of several existing high-performance minicomputer models would be capable of handling all IMIS functions. Furthermore, if improvements in hardware and software continue at the same pace as in recent years, the capability of minicomputers expected to be available at the time of IMIS implementation will far exceed the requirements.

The computer configuration recommended for the more versatile of the alternative system designs consists of two separate minicomputers and data communications units to provide essentially continuous operation in the event of equipment failure and during periods of preventive maintenance. This configuration also permits program modification and debugging on the off-line computer without requiring a foreground/background operating system. To provide maximum system reliability, manual switching would be used to switch the computers and communications units between the on-line and off-line states. Low-speed inter-computer communications would provide essential system status information to the off-line computer to minimize the period of system standby operation during the transition.

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*Sperry Systems Management, "Development of Traffic Logic for Optimizing Traffic Flow in an Intercity Corridor", Contract DOT-FH-11-8738, April 1976

Each computer would consist of the following components:

- Central Processing Unit with 320,000 words of main memory and input/output interface
- Disk memory
- Keyboard/printer terminal
- Video (CRT) terminal
- Card reader
- Line printer
- Magnetic tape unit

The central data communication unit for the control center would consist of a modem (modulator/demodulator) connected to each voiceband channel and interfacing with the computer through the computer's serial/parallel converter units. For transmitting data, the modems receive commands and field cabinet address data from the computer in serial form and convert it to a modulated waveform (frequency-shift-keyed or phase-shift-keyed) suitable for transmission. For reception, the modem receives modulated data from the transmission medium and converts it to serial dc waveforms for entry into the computer.

The IMIS control center should also include a dynamic map, consisting of a wall-mounted graphic display containing color-coded computer-driven lamps at each detector station, traffic signal, and variable-message sign location. Threshold controls then permit use of the detector display to indicate traffic problems, while signal and sign displays may be used to indicate their respective operating status. Computer-detected failure of field equipment is also normally displayed by the lamps.

13.5.2 Control Center Operation

Personnel operating the IMIS control center will be required to perform the following tasks:

- Monitor computer outputs (on map display, CRT terminal, and printers) and computer decisions.
- Respond to computer outputs as required, by either manually over-riding computer decisions or contacting other agencies (e.g., police)
- Note indications of central or field equipment failure and either call for maintenance action or switch to off-line computer, as required

- Respond to telephone calls from police, other local agencies, and the public

- Modify system functions and/or associated software to incorporate improvements or remedy defects

- Add new messages to library tape for roadside radio as required

- Perform administrative duties

- Supervise overall operation of system

For estimating the personnel requirements it should be assumed that the more versatile of the alternative system designs will be manned full time (24 hours a day, 7 days per week). The amount of activity will vary at different times of day, day of the week, and season, resulting in variations in personnel requirements at the center.

For estimating purposes, the following may be considered as a typical staff:

Day	Period	Position	Coverage
Weekday	Std. day shift	Supervisor*	1
Weekday	Std. day shift	Programmer	1
Weekday	Std. day shift	Traffic Operations Eng.	3
Weekday	2nd and 3rd shifts	Traffic Operations Eng.	2
Saturday and Sunday	1st and 2nd shifts	Traffic Operations Eng.	2
Saturday and Sunday	3rd shift	Traffic Operations Eng.	1

The above coverage amounts to 22,880 man-hours per year. Assuming an average of 1920 hours worked per year per person (48 weeks), a total of about 12 employees will be required.

13.5.3 Cost Estimates

The control center costs consist of (1) an initial capital cost for equipment, installation and checkout, (2) an annual cost for equipment maintenance and (3) an annual operating cost. Typical estimates for these items are summarized below, except for average labor rates and overheads which should be estimated by the user.

----

*Also on call for emergencies

- Capital Costs

  Cost of two computers and associated peripherals, map display, control panel, communications, cabling, room preparation, air conditioning, installation, and checkout $550K

- Annual Maintenance Costs

  Taken as 12 percent of the capital cost. $66K/yr.

- Annual Operating Costs

  Personnel: 12 people x average annual salary (including overhead)

- Telephone facilities, miscellaneous consumables, room power $20K/yr.

For lower cost system alternatives, the following may be considered:

- Use of a single computer - This will reduce the capital costs to approximately $310K, maintenance costs to about $37K/yr, and operating costs to about $17K/yr.

- Half-time manned operation (e.g. 6 A.M. to 6 P.M.) - This will reduce personnel operating costs to approximately 65% of the full time costs.

It has been assumed in the foregoing that sufficient floor space (minimum of 1,000 square feet (93 square meters)) can be made available to serve as the IMIS control center. Thus, only room preparation costs have been included. If this is not the case, appropriate additional costs must be added.

13.5.4 Control Center Guidelines

For purposes of developing the alternative designs, the most versatile system should have the full coverage staffing level (i.e. 24 hours per day, 7 days per week). Any of the reduced complement designs within each network could be set to a half coverage level.

For the most versatile system, the dual computer configuration should be selected. The single computer can be specified for any of the lower cost alternatives.

13.6 OUTPUTS OF THE SYSTEMS DESIGN TASK

As shown earlier in Figure 17 there are two outputs of the alternative system design procedure. The network maps with field equipment stations indicated provides the spatial relationships between the several subsystems, and between the subsystems and the corridor. In this way the physical differences of the system

designs can be related to the several network configurations. The system summary table gives the equipment levels of each subsystem for each IMIS design. Table 42 gives a sample layout of this summary table for a group of nine alternative designs divided into 3 subgroups (network configurations) with 3 subsystems design complements in each subgroup. The two dimensional design process (Figure 16) is clearly shown within this group/subgroup arrangement. The table elements are the equipment quantities for each subsystem for which unit costs were defined.

Table 42. Example of System Design Summary Table

Subsystem	Equipment Complement	Candidate Designs								
		Network Configuration A			Network Configuration B			Network Configuration C		
		A1	A2	A3	B1	B2	B3	C1	C2	C3
Diversion/Motorist Advisory	Visual Sign Loc.	75	55	25	70	50	25	65	45	12
	Hwy Adv. Radio Loc.	17	26	20	6	26	20	3	23	10
Ramp Metering	Metering Stations	72	72	57	72	72	57	72	72	57
Arterial Control	CIC Intersection	36	36	36	37	37	37	6	6	6
	Non-CIC Int.	65	65	65	58	58	58	10	10	10
Surveillance	Freeway-Det Spacing (Miles)	1/2	1/2	1/2	1/2	1/2	1	1/2	1/2	1
	Arterial # Det Sta. (Non-CIC)	130	130	130	116	116	116	20	20	20
Communications	Cabinet Loc.-#	800	800	800	640	640	520	640	640	520
Control Center	Computers - #	2	2	1	2	1	1	1	1	1
	Coverage - %	100	100	50	100	50	50	50	50	50
Motorist Aid	Stations	230	0	0	230	0	0	230	0	0
Pre-Trip Information Serv.	Inclusion	YES	YES	NO	YES	YES	NO	YES	YES	NO
Incident Management	Police Cars - #	1	0	0	1	0	0	0	0	0
	Tow Trucks - #	2	2	0	2	2	0	2	0	0

Note: 1 mile = 1.61 Km

CHAPTER 14

DETERMINATION OF SYSTEM COSTS

14.1 INTRODUCTION

14.1.1 Objectives

- To compute the capital, maintenance, and operating costs for each subsystem of the candidate designs.
- To estimate associated implementation costs.
- To compute total equivalent annual costs for each candidate design.

14.1.2 Inputs

- Candidate IMIS designs as defined by system design summary table (Chapter 13)
- Unit costs for the IMIS field based subsystems (Chapter 12) and costs for the communications subsystem and control center (Chapter 13)

14.1.3 Outputs

- Equivalent annual costs for each of the candidate system designs

14.2 PROCEDURE

The cost associated with a candidate design can be classified into two broad categories - recurring costs and nonrecurring costs. The nonrecurring costs refer to the costs associated with design of the system, the purchase of equipment, and the construction and installation of the system. Recurring costs are maintenance and operating costs. Figure 19 shows how the cost components are assembled to achieve a total system cost.

The cost components of capital equipment/construction, maintenance and operations were discussed previously. Maintenance of traffic is, as the name implies, the cost associated with maintaining and protecting traffic on the roadways during the construction phase. For major roadway projects requiring installation of equipment in the traveled way, an estimate of about 5 percent of the system hardware and construction cost may be used if more specific costs are not available to the using agency.

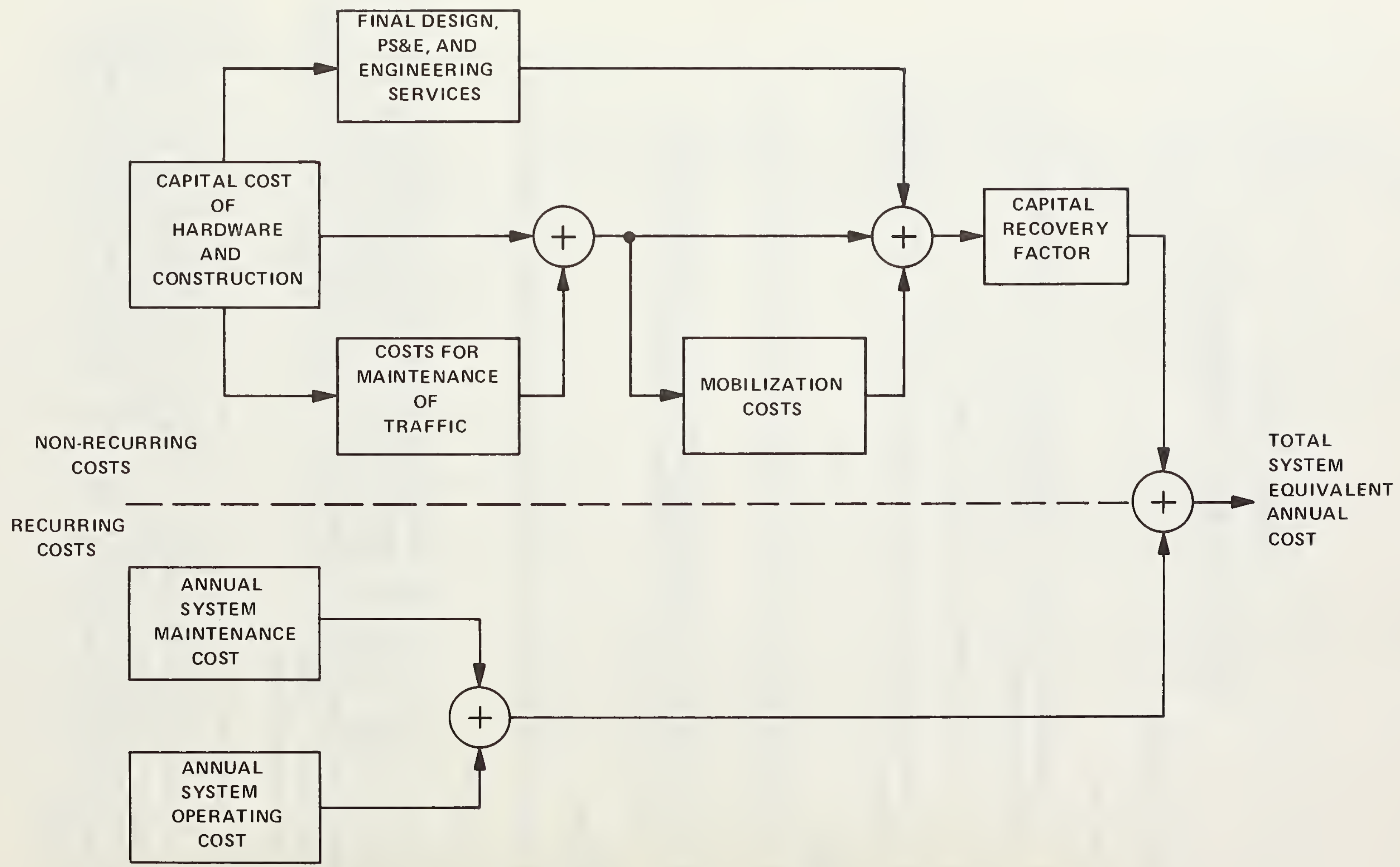

Figure 19. System Cost Determination Procedure

Mobilization is the component of the total effort which consists of the operations preparatory to actual construction. These operations typically consist of the movement to the project site of personnel, equipment supplies and incidentals necessary to begin work on the project. This item is estimated as a percentage of hardware and construction cost (including maintenance of traffic cost), typically on the order of 3 percent.

The final category of nonrecurring costs are the costs associated with the engineering design, generation of the detail plans, specification and cost estimates (PS&E) and engineering services associated with system implementation. The engineering costs for computer-based traffic surveillance and control systems is typically higher than the cost of similarly priced highway construction contracts. This is necessitated by the number of specialized disciplines that are required for the work including communications engineering, computer and software design engineering, system engineering, and traffic engineering. Similarly, project management tends to be more complex due to such items as subsystem and system test, inspection at roadside, and administration of the contracts. Typically, therefore, while highway construction engineering costs vary from 5 to 10 percent of hardware and construction costs, surveillance and control system engineering costs can be expected to vary from 10 to 20 percent. For systems at the level of complexity of IMIS it is recommended that at least 15 percent be used as the estimate, based on an approximate breakdown of 6 percent for final design/PS&E, and 9 percent for engineering services. It is noted that engineering services include software development and computer programming costs.

To obtain an equivalent annual cost for the nonrecurring cost elements, they are added together and a capital recovery factor is applied to the total. The capital recovery factor is a function of the expected system life and interest rate. Figure 20 shows capital recovery factors for system life and interest rates common to these projects. (Actual factors may be found in most Economics textbooks). Final selection of a factor would be based on the interest rate and system life utilized by the cognizant agencies for evaluation of similar projects. The equivalent annual non-recurring cost is obtained by multiplying the total cost by the capital recovery factor.

Table 43 provides a sample worksheet for generating the IMIS cost on an equivalent annual basis. Each candidate system should have a separate worksheet. The general procedure for assembling these worksheets is to multiply the basic unit cost data developed earlier for each IMIS subsystem by the equipment quantities specified in the System Design Summary table (Table 42). These subsystem costs are added to obtain separate capital, maintenance, and operation total costs. The remaining nonrecurring cost factors are obtained using the equations given in the Table 43. The equivalent annual total system cost is then obtained by summing the nonrecurring total annual cost (F') with system maintenance (G1) and operation (G2) costs.

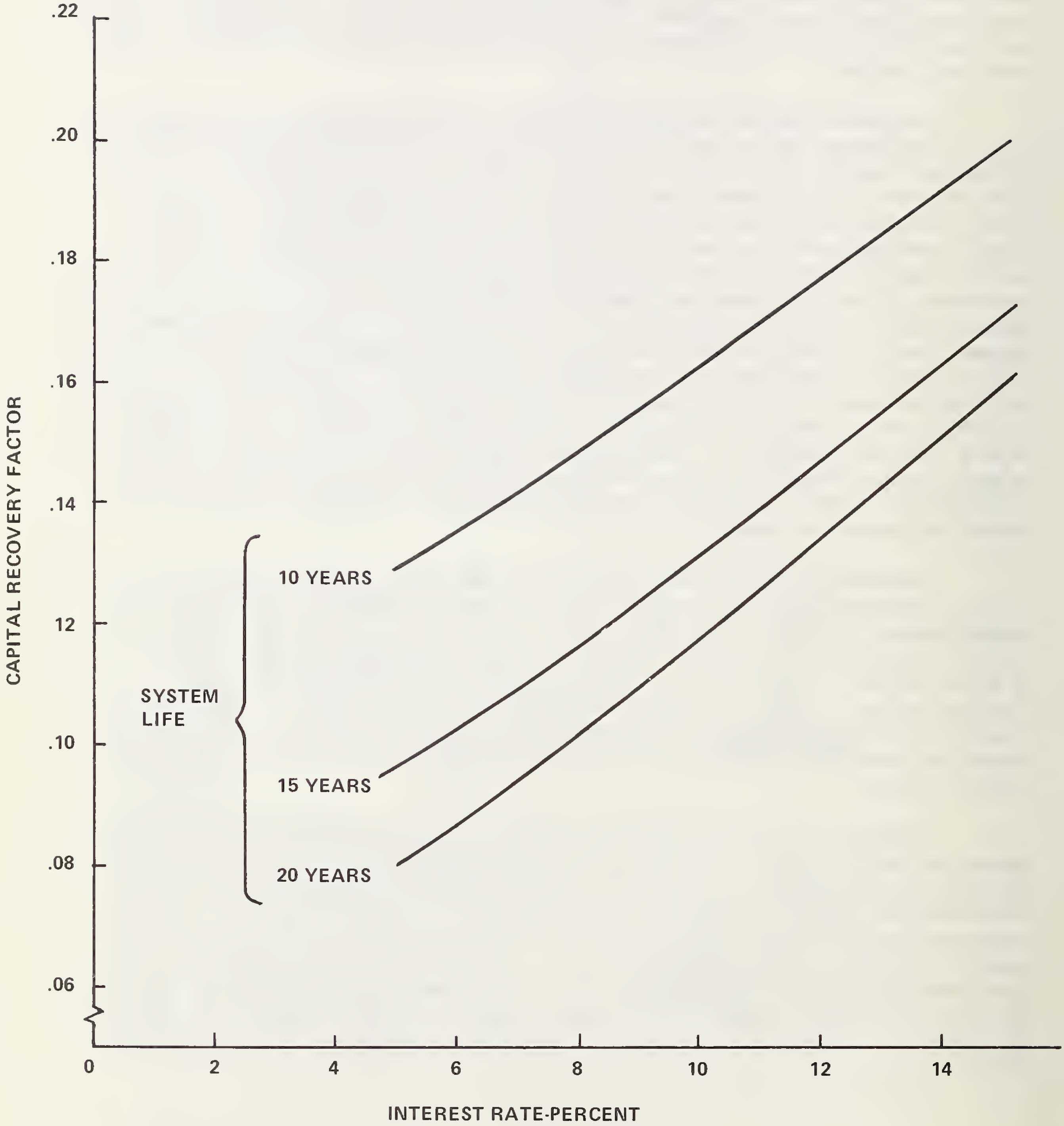

Figure 20. Capital Recovery Factors

Table 43. Sample Worksheet, Candidate System Cost

Cost Factor	System Cost		
	Capital (M$)	Annual Maintenance (K$)	Annual Operation (K$)
A. Subsystem 1. Variable Message Signs 2. Highway Advisory Radio 3. Ramp Metering 4. Freeway Surveillance 5. Arterial Surveillance 6. Communications 7. Equipment Cabinets 8. Control Center 9. Motorist Aid 10. Pre-Trip Information 11. Miscellaneous*			
B. TOTAL - ALL SUBSYSTEMS CAPITAL			
C. MAINTENANCE OF TRAFFIC C = (.05) (B)			
C. MOBILIZATION D = (.03) (B + C)			
E. FINAL DESIGN, PS&E ENGINEERING SERVICES E = (.15) (B)			
F. NONRECURRING TOTAL F = B + C + D + E F' = Equiv. Annual Value of F			
G. RECURRING TOTAL 1. Maintenance 2. Operation			
H. SYSTEM TOTAL (EQUIV. ANNUAL) H = F' + G1 + G2			

*Includes fixed message signs, other system surveillance

CHAPTER 15

DETERMINATION OF SYSTEM BENEFITS

15.1 INTRODUCTION

15.1.1 Objectives

- To develop an estimate of the system benefits which accrue to each IMIS candidate design when implemented in a given corridor. Benefit categories of delay, fuel consumption, accidents and air pollution are explicitly considered.

15.1.2 Inputs

- Outputs of Chapters 6, 7 - Basic flow, accidents and operational characteristics of corridor roadways.

- Outputs of Chapters 9, 10 - Network configurations, control subnetworks and probability factors.

- Output of Chapters 12 - Set of candidate designs including field equipment maps and summary table.

15.1.3 Outputs

- Benefit worksheet for each design candidate. Worksheet shall include categories of delay, fuel consumption, accidents and air polution.

15.2 OVERVIEW OF COMPUTATIONAL PROCEDURE

The entire computational effort required to obtain a total benefit for each system candidate is conducted in this task. Each benefit category is considered in the sequence: vehicle delay, accident reduction, fuel consumption and pollutant emission.

Figure 21 presents the computational flow diagram for the entire benefit task. The major categories of inputs to the vehicle delay blocks (1, 2, 3) are the roadway configuration (from Chapter 9), the control and equipment complements (from Chapters 10, 12) and roadway flow characteristics including incident/accident rates (from Chapter 7). The procedure for the delay benefit computation is to develop a fundamental benefit factor referenced to a single lane-mile of corridor roadway. As a function of the roadway flow characteristics and of the layout of the control area boundaries, distinct benefit factors are assigned to specific subnetworks of the corridor. The total delay benefit is obtained by mulitplication of the fundamental benefit factors by the number of lane-miles of roadway within the subnetwork of corridor.

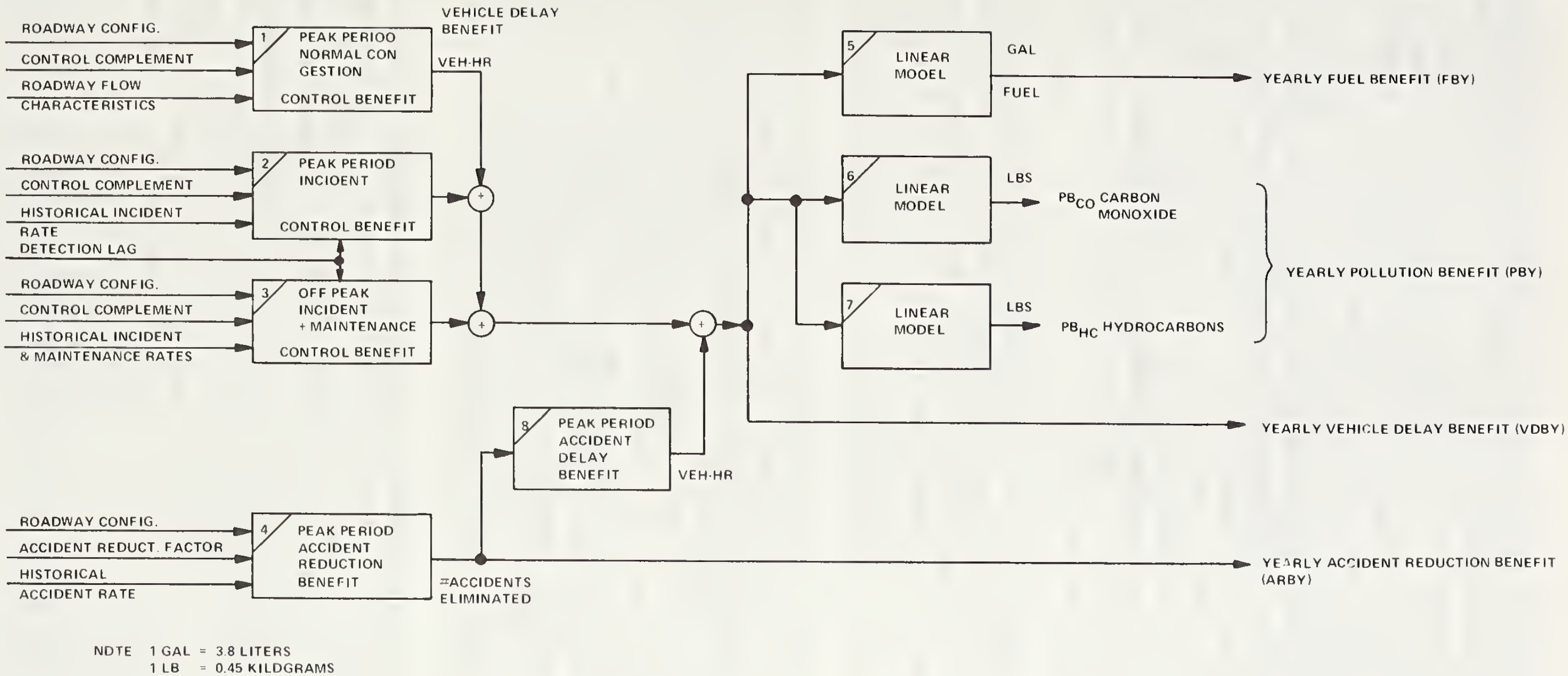

Figure 21. IMIS Benefit Methodology - Computational Flow Diagram

The computation of the fuel consumption benefit (block 5) is obtained by utilizing a linear relationship between vehicle delay and excess fuel consumed. In a similar manner, pollutants emitted (blocks 6, 7) are also calculated using a linear relationship to vehicle delay. The accident reduction benefit, blocks 4, 8, is computed by determining an accident rate for the limited access roadways in the corridor. This rate is defined with respect to a lane-mile of roadway. Based on documented before and after studies of operational surveillance and control systems an accident reduction factor is applied to this historical rate. Finally multiplying by the lane-miles of roadway in the corridor quantifies the accident reduction benefit. This accident reduction contributes to overall benefits in two ways. First, there is a reduction in annual accident costs which is a direct safety benefit, and second there is reduction in vehicle-hours of delay. This accident delay benefit is not related to the peak or off-peak incident benefit since that benefit accrues to the user after an accident has occurred. The accident delay benefit accrues to the user because the occurrence of the accident has been eliminated.

The following paragraphs discuss the detailed computational aspects of each benefit category. Particular attention should be devoted to the procedures outlined for peak-period normal congestion and incident delay categories since these areas typically develop about 75 percent of the total benefits derived from system operation.

15.3 DELAY BENEFIT COMPUTATION - PEAK PERIOD NORMAL CONGESTION

The peak period normal congestion benefit addresses the requirement of IMIS to reduce the level of recurrent traffic congestion which occurs at various corridor locations on a daily peak period basis. The control concept implemented in order to realize these benefits shifts demand volumes from roadways which are experiencing congestion to roadways which have available capacity. Depending upon the geometrical/physical configuration of the roadways, the shift of demand can be implemented with the control functions of ramp metering or traffic diversion utilized separately or in combination. An important factor which is accounted for by the computational procedure is the benefit-disbenefit tradeoff relationship which exists between the roadway from which traffic is shifted and the roadway which received the additional traffic. The incorporation of this factor is influenced by the configuration of the roadways which are involved in the shift of traffic volumes.

The three generic roadway configurations (subnetworks) were specified in Chapter 10. For each of these three subnetwork types, a fundamental normal congestion delay benefit (net benefit after accounting for any disbenefit) can be developed. This benefit is referenced to a lane-mile of corridor per hour of peak period operation. Table 44 presents the individual benefit and disbenefit equations which when properly combined provide the net benefit for each network type. Figure 22 provides an overview of the computational procedures. Two factors should be noted from this figure: first that the benefit and disbenefit equations are combined uniquely for each subnetwork type, second, that the roadway conditions which exist in each section must be specified for each subnetwork type.

The benefit equation is based on the concept that actual benefits may be expressed as some fraction of the theoretical maximum benefit. The theoretical maximum benefit is that which would be achieved if all delay was eliminated; it is

thus equal in magnitude to the total delay. The theoretical maximum benefit, D_{SS}, may therefore be calculated from the following expression:

$$D_{SS} = \overline{Q}_{PP} \left[\frac{(U_{FF} - U_C)}{(U_{FF})\ (U_C)} \right] \tag{15}$$

Table 44. Fundamental Benefit and Disbenefit Relationship* Peak Period Normal Congestion.

BENEFIT EQUATION

$$VDB = D_{SS} \left[(1 - e^{-\alpha \Delta Q}) - \beta\ (\alpha \Delta Q)\ e^{-\alpha \Delta Q} \right] \tag{16}$$

DISBENEFIT EQUATION

$$DS = (K_x)\ (DS1) + (K_x - 1) \left(\frac{\Delta Q}{U_c} \right) \tag{17}$$

WHERE: $$DS1 = \frac{(U_c - U_A)}{(U_c)\ (U_A)} \Delta Q + S_D\ \Delta T_{DA}\ (QA + \Delta Q) \tag{18}$$

DEFINITIONS:

D_{SS}	-	Theoretical Maximum Benefit = (Equation 15)
ΔQ	-	Control Volume Shift
α	-	Volume Shift Factor (nominal value = 0.01)
β	-	Congestion Severity Factor - (function of roadway speed)
K_x	-	Alternate Route Distance Penalty Factor
S_D	-	Density of Signals on Alternate Route
ΔT_{DA}	-	Travel Delay per Vehicle per Signalized Intersection
U	-	Roadway Speed, FF - Free Flow on Freeway C - Congested on Freeway A - Arterial

* The development of these generalized relationships was based in part on the extensive simulation work performed during the Long Island IMIS feasibility study. The equations were derived by curve-fitting the simulation data points.

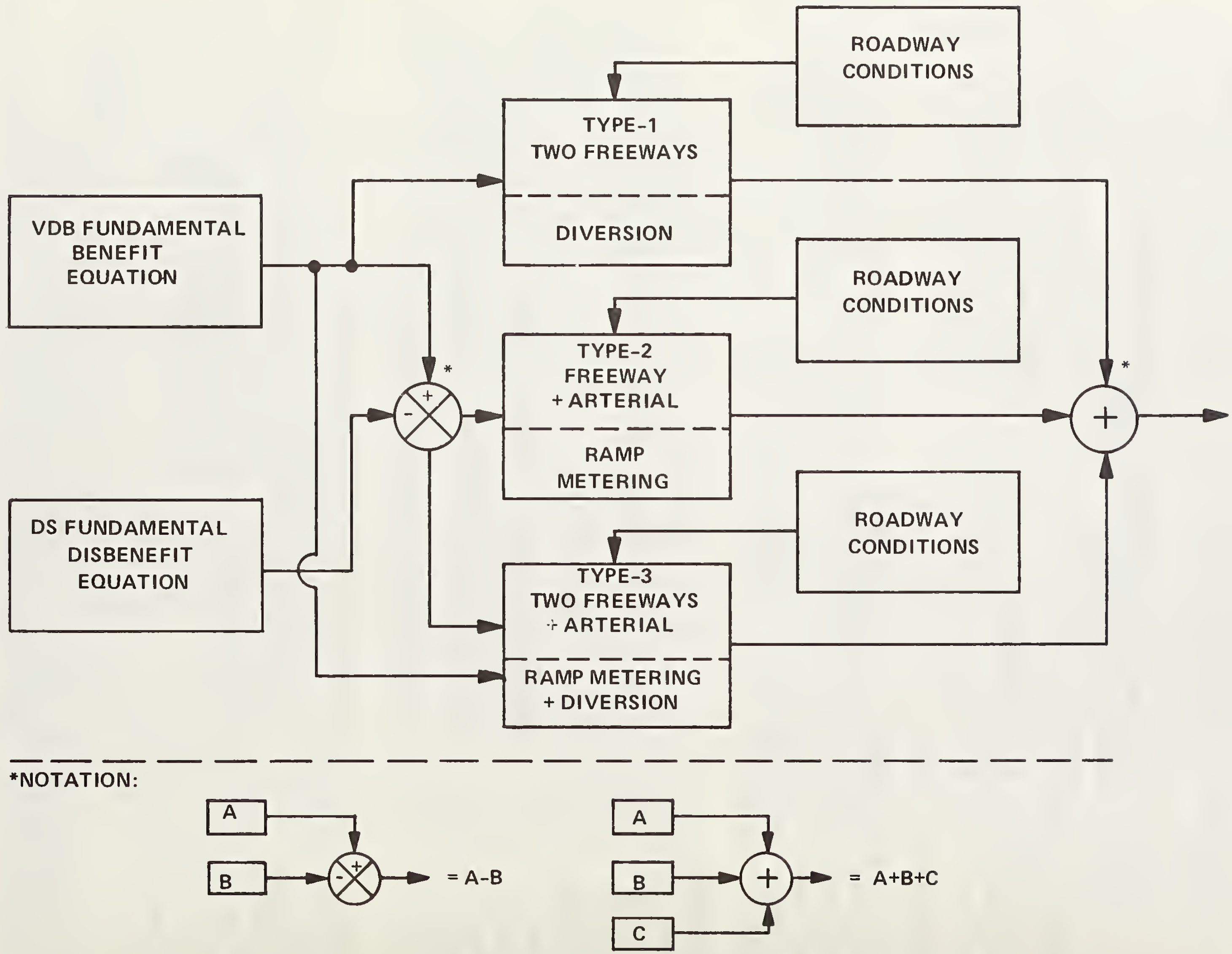

Figure 22. Computational Procedures for Peak Period Normal Congestion

where $\overline{Q}_{PP}$ is the peak hour mean flow, and U_C is the peak hour mean speed. The quantity U_{FF} is the speed of the vehicles in the traffic stream when they are experiencing zero delay. This is typically equal to the vehicle speed during free flow conditions. A speed of 55 mph (88.6 KPH) is a reasonable assumption for limited access freeways since this is the national speed limit. It is noted that achievable benefits are quite sensitive to the values of $\overline{Q}_{PP}$, U_C, and U_{FF}, since these three characteristics in effect set the upper bound on the benefit levels.

The exponential term of the benefit equation (16) in Table 44 within brackets is intepreted as that fraction of the theoretical benefit which represents an actual improvement in corridor operation due to an IMIS system with a control volume shift capability of ΔQ. Two parameters α and β determine the shape of the exponential curve. The nominal value of α is .01. β is dependent on U_C and is obtained from Figure 23. (These values for α and β were developed during the Long Island IMIS study.) Figure 24 shows a set of benefit curves based on the stated set of corridor parameters. If the standard values of $\overline{Q}_{PP}$ or U_{FF} are not considered appropriate by the user, the user should determine the benefit levels from the original equations by inserting appropriate values determined from the site data base.

The fundamental benefit information contained on Figure 24 can be used without modification to obtain benefits for the roadway subnetwork type 1. For roadway subnetwork types 2 and 3, since there is a shift or diversion of traffic to a non-freeway type facility, the benefit information is modified to account for the disbenefit experienced by the vehicles on the alternate roadway. This disbenefit factor is composed of three parts. The first component

$$K_x \frac{\left(U_C - U_A\right)}{\left(U_C \quad U_A\right)} \Delta Q$$

is a disbenefit which quantifies the observation that the vehicles shifted to the alternate would travel on the arterial at a mean speed U_A which is in general different from the mean freeway congested speed U_c. If U_A is less than U_C, $(U_C - U_A)$ positive, the motorist receives a disbenefit; if U_A is greater than U_C, $(U_C - U_A)$ negative, the diverted motorists receive a benefit

The second component

$$(K_x - 1) \frac{\Delta Q}{U_C}$$

is a disbenefit which quantifies the observation that the motorists diverted to the alternate would in general travel an increased distance. The third component

$$K_x \, S_D \quad \Delta T_{DA} \left(Q_A + \Delta Q\right)$$

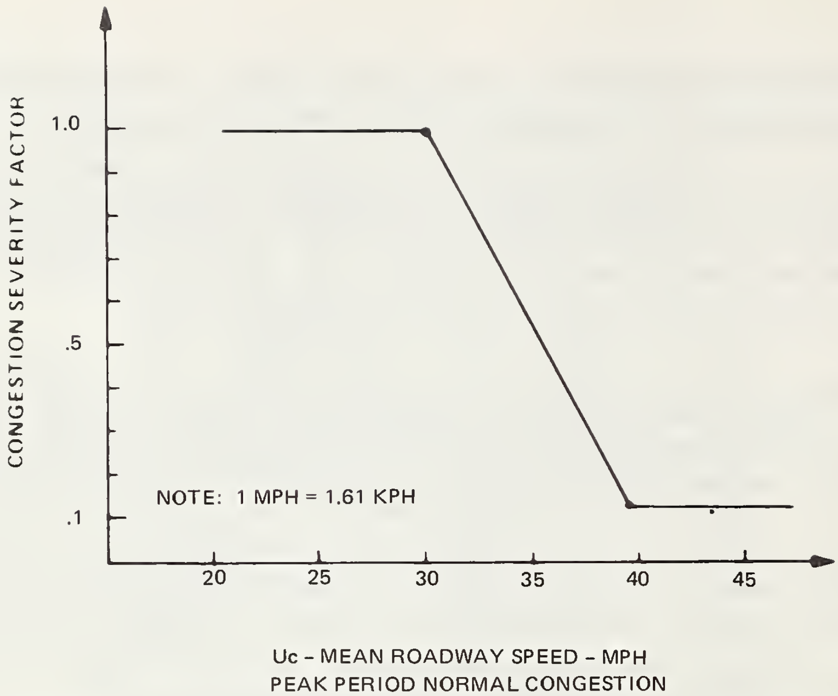

Figure 23. Congestion Severity Factor for Benefit Equations

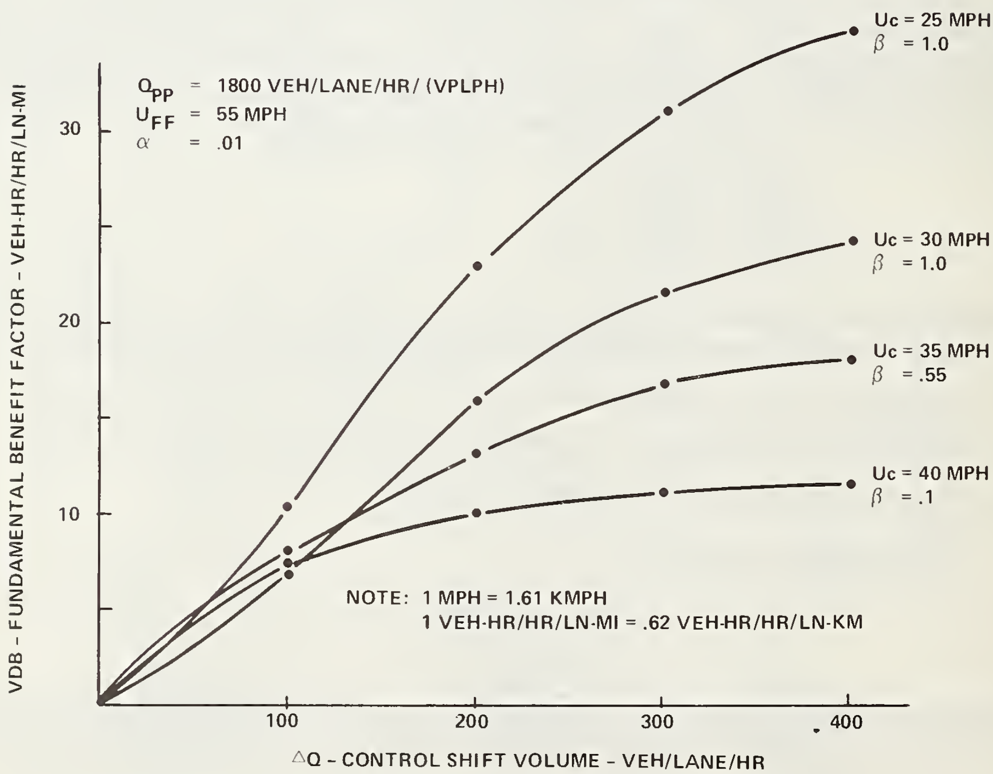

Figure 24. Typical Benefit Curves Type 1 Subnetwork
Peak Period Normal Congestion

is a disbenefit which quantifies the observation that the motorists diverted to the alternate would increase the total traffic demand at the signalized arterial intersections and hence would increase the mean delay per vehicle which is experienced by all vehicles moving through the intersections. The magnitude of this third component is influenced by the presence on the alternate of an arterial signal system with the capability to respond to the changes in demand levels. The dependence is realized through the term ΔT_{DA} (incremental delay per vehicle per signalized intersection). This term is directly related to the interaction of the signal parameters (cycle, split, offset) and the normal arterial demand level (Q_A). Table 45 presents the defining equation for this incremental delay term.

Table 45. Incremental Arterial Delay Per Signalized Intersection (ΔT_{DA})

$$\Delta T_{DA} = \frac{A}{S\left(1 - \frac{Q_A}{S}\right)^2} + \frac{B}{\left(1 - \frac{C}{GS} Q_A\right)} + \frac{B \frac{CQ_A}{GS}}{\left(1 - \frac{C}{GS} Q_A\right)^2} \tag{19}$$

$$A = .45\, C \left(1 - \frac{G}{C}\right)^2 \tag{20}$$

$$B = .45 \left(\frac{C}{GS}\right)^2 \tag{21}$$

DEFINITIONS:

ΔT_{DA}	-	Delay per vehicle at intersection - SEC/VEH
G	-	Nominal green time assigned to arterial flow - SEC/CYCLE (Select signal which acts as bottleneck)
C	-	Nominal cycle length assigned to arterial signals - SEC (Select signal which acts as bottleneck)
S	-	Saturation flow through intersection during green interval - VEH/LN/SEC; typical value = .5 VEH/SEC, (equivalent to 1800 VEH/HR)
Q_A	-	Nominal flow along arterial - VEH/LN/SEC

The fundamental benefit information for roadway subnetwork type 2 is given in Figures 25 and 26. The typical traffic parameters are presented on the figures. Figure 25 is used when the alternate roadway is a service road and Figure 26 when the alternate is a parallel arterial. The difference between these two figures is based on the distance penalty which a motorist diverted to an arterial must accept in comparison to a diversion to a service road. The service road runs directly parallel to the freeway facility and hence there is no distance penalty. The arterial, with its own right of way clearly separate from other facilities, almost always imposes a distance penalty on motorists diverted to it from the primary facility. For Figure 26 the distance penalty has been set to 1.25 as a typical value. The figures were also

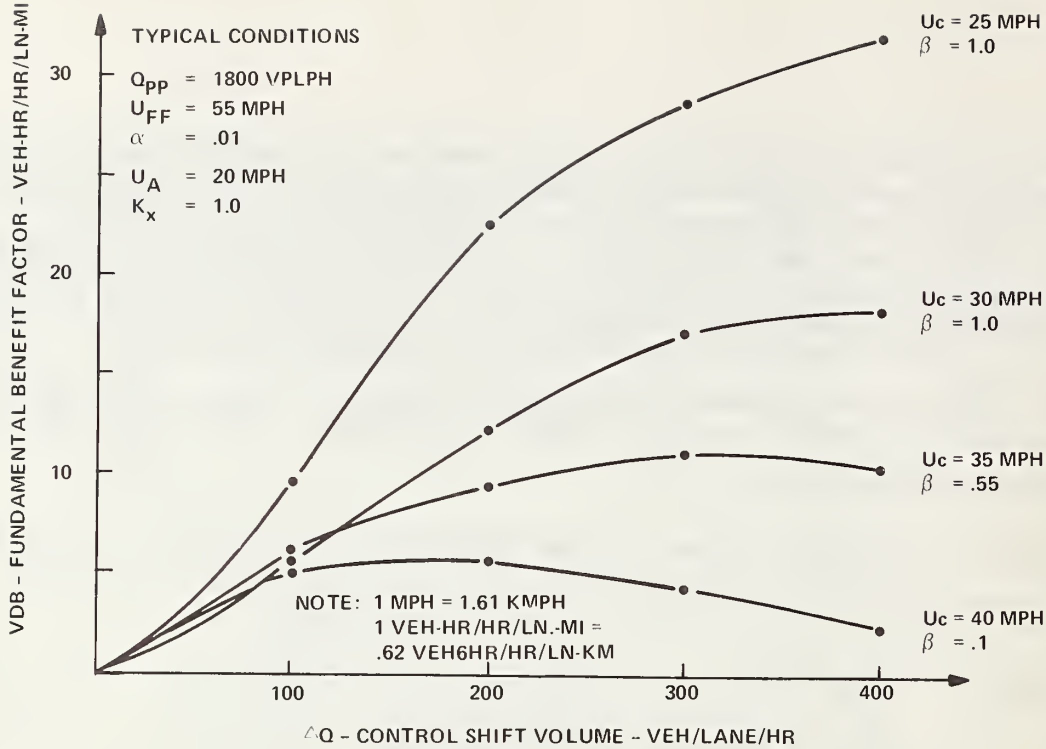

Figure 25. Typical Benefit Curves, Type 2 Subnetwork - Service Road Peak Period Normal Congestion

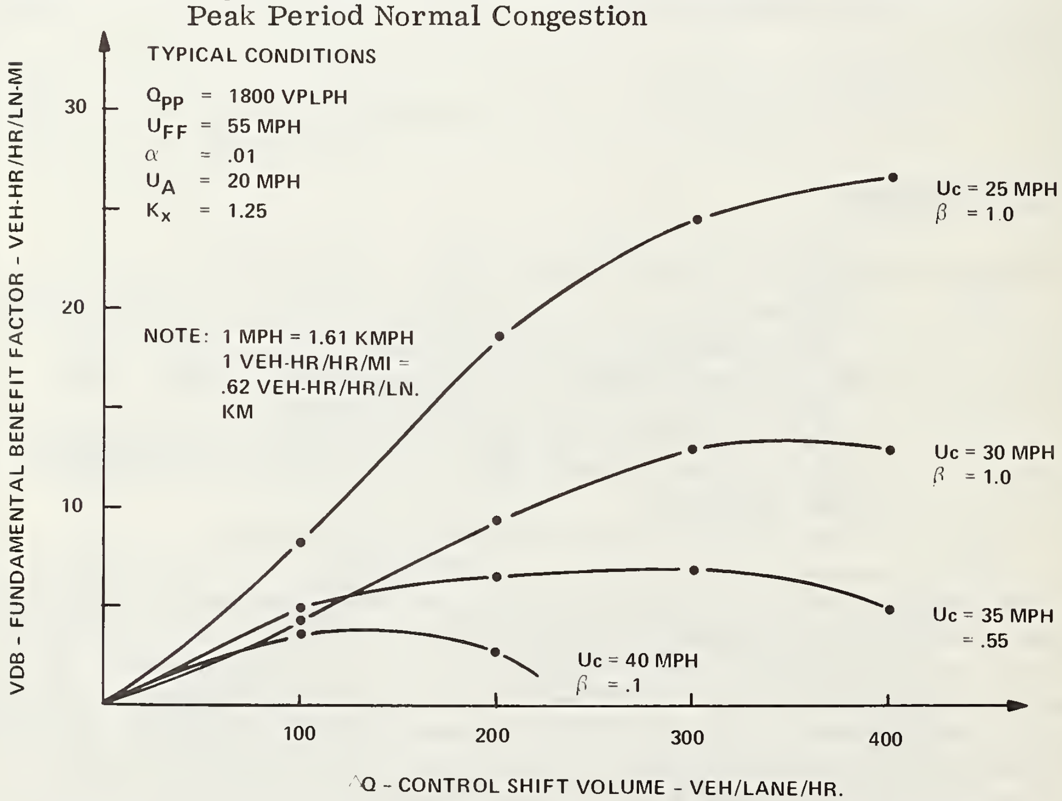

Figure 26. Typical Benefit Curves, Type 2 Subnetwork - Arterial Peak Period Normal Congestion

constructed with the assumption that the service road or arterial is operating under traffic responsive signal control. This assumption allows the third component of disbenefit to be assumed zero since the timing plan can be altered in response to the increased demand of the diverted vehicles.

The fundamental benefit value for roadway subnetwork type 3 is determined from a weighted linear combination of the benefits contained in Figure 24 (type 1) with those in Figure 25 or 26 (type 2, service road or arterial, respectively). For a given set of roadway conditions and level of control (control volume shift), the component benefits are assigned equal weight (each 0.5). The assignment of equal weights to each component reflects the observation that on the average, traffic diversion will be split between the freeways and from the freeways to the service road and/or arterials.

The following subsections describe the specific computational steps for determining the delay benefits for peak period normal congestion. Initially, the computation is performed for the most versatile system (subsection 15.3.1). Then, the benefit computations for the remaining alternative system designs are addressed (subsection 15.3.2).

15.3.1 Computation Procedure for Most Versatile System

The computational procedure for the yearly peak period normal congestion benefit consists of a sequence of six steps.

(1) Establish the level of control volume shift (ΔQ) for each subnetwork. For the freeways, the value is obtained from the control volume shift calculation of Chapter 10. For the arterials, the value is based on the average of the available capacities developed in Chapter 7.

For example, assume a given subnetwork consists of 2 freeways and 2 alternate arterial routes, and the following results were obtained earlier in Chapters 10 and 7:

Freeway A – ΔQ = 150 veh/hr/lane
Freeway B – ΔQ = 150 veh/hr/lane
Arterial A Rte Avg. available capacity = 95 veh/hr/lane
Arterial B Rte Avg. available capacity = 80 veh/hr/lane

The level of control possible for a diversion from freeway A (or B) would then be the square root of the sum of the squares (rss) of the other roadway valves, i.e., $(150^2 + 95^2 + 80^2)^{1/2}$ or 195 veh/hr/lane. The "rss" valve is used (instead of a straight sum) to account for random variations in the quantities.

(2) For each subnetwork, the fundamental benefit level is determined by using the appropriate figures 24 thru 26. The benefit level will be determined by the mean roadway speed Uc appropriate to each set of roadways.

(3) Multiply the fundamental benefit levels by the number of lane-miles of each subnetwork.

(4) Multiply the output of step (3) by the appropriate control probability factor developed in Chapter 10. See Table 32 for the typical values of this factor.

(5) Multiply the output of step (4) by the benefit expansion factors to obtain a yearly delay benefit for peak period normal congestion category. Table 46 defines the set of expansion factors for the normal congestion benefits computation.

(6) Sum the yearly benefits of each subnetwork developed in step (5). This final value is the delay benefit derived from yearly operation of the most versatile system during peak travel periods.

Table 46. Benefit Expansion Factors, Peak Period Normal Congestion

Factors	Description
K1 - Peak Period Duration	Duration of a weekday peak period. This factor should be estimated for each corridor.
K2 - Peak Period Number	Number of peak periods per day during which system is used. Typical value is 2 per day.
K3 - Number of Days/Year	Number of days in a year considered for benefit computation. Typical value is 250 days.
K4 - Peak Period Direction	Number of traffic flow directions to be considered during a time period. Typical value is 1 direction.

15.3.2 Computations for Remaining Alternative Systems Designs

The development of a yearly peak period normal congestion delay benefit for each of the remaining candidate systems is found by relating the design variations to the fundamental benefit factors of Figures 24 and 25 and the number of lane-miles of corridor roadways which is covered by each design. The design variations which provide the major influence on the generation of corridor-wide system benefits are the elimination of alternate routes (as defined by the reduced roadway networks) and reductions in the number of control (diversion) points.

Elimination of alternate routes will impact the level of control (ΔQ) that can be utilized during system operation (i.e., the effectiveness of control is reduced). In this case, the eliminated routes are simply excluded from the "rss" computation of the total ΔQ. In addition, elimination of routes may change sections of the corridor from subnetwork types 2 or 3 to types 1 or 2. Where this is the case, appropriately revised control probability factors must be used.

System designs with a reduced number of control points contain less diversion flexibility than their corresponding maximum performance systems. Thus it would be expected that their achievable control volume shift capabilities would be reduced. To estimate the reduced effectiveness of having fewer diversion points (diversion sign locations) compared to the maximum performance system, the following equations may be used:*

$$\Delta Q_2 = R \Delta Q_1 \tag{22}$$

$$R = \frac{n_2 \left(1 - e^{-\lambda \frac{C}{n_2}}\right)\left(e^{-\frac{C}{n_2 D}}\right)}{n_1 \left(1 - e^{-\lambda \frac{C}{n_1}}\right)\left(e^{-\frac{C}{n_1 D}}\right)} \tag{23}$$

$$\lambda = (.693)/MTL \tag{24}$$

where

ΔQ_2 = control volume shift capability of the less effective system

ΔQ_1 = control volume shift capability of the maximum performance system

R = efficiency of reduced sign complement with respect to full maximum sign complement (fraction of control volume capability remaining for less versatile system designs)

C = overall length of corridor

n_i = number of sign stations per limited access roadway direction for ith system

$i = 1$ for maximum capability system
$i = 2$ for next less versatile system

λ = loss rate of traffic per unit distance

MTL = median trip length for corridor

D = relevency distance = 12 miles (19.3 km)

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*The derivation of the equations and further definitions of terms may be found in Appendix G.

For candidate designs for which highway advisory radio (HAR) stations are substituted for visual signing stations, the effectiveness of HAR is assumed to be 75 percent of that of a visual sign.*

The final steps for computation of the total normal congestion delay benefit of each alternative system candidate are identical to steps (2) through (6) enumerated previously for the most versatile system candidate.

15.4 DELAY BENEFIT COMPUTATION - PEAK PERIOD INCIDENT

The peak period incident benefit addresses the requirement of IMIS to reduce the magnitude, extent and disruptive consequences of incident caused congestion. The control concept implemented in order to realize these benefits is to shift a portion of the traffic demand approaching an incident site to alternate routes selected on the basis of their ability to handle the additional load and maintain acceptable operations.

The key steps of the computational procedure parallel in many aspects the normal congestion procedure given in Section 15.3. In particular the use of the roadway configurations, the benefit/disbenefit tradeoff, the control probability factors, and benefit expansion factors are required by the procedures given in this section.

The development of the incident benefit is based on the Incident Delay Diagram as shown in Figure 27. The abscissa specifies the time evolution of the incident scenario. The oridinate specifies the number of vehicles who desire to utilize the roadway (demand curve) and those who actually utilize the roadway at the location of the incident (capacity curve). See Table 47 for definitions of the parameters used in the figure. The incident scenario is composed of two distinct time lines corresponding to incident occurrence without and with control. For the scenario with control, the demand curve is modified by the shift of a fraction of the demand volume to an alternate roadway within the corridor. Referring to the figure, the standard benefit from IMIS control of a single incident is equivalent to the area between the demand curves.

The defining equation for incident benefit is based on the difference between the levels of incident delay generated without and with control. The incident delay is in turn defined by the difference in areas under the appropriate demand and capacity curves. Referring to Figure 27 the delay equations are:

$$\mathrm{DELAY}\Big|_{\mathrm{W/O\,CTL}} = \int_{0}^{T_{CC1}} N_D(t)\,dt - \int_{0}^{T_{CC1}} N_C(t)dt \qquad (25)$$

$$= NDI_1 - NCI_1$$

----

*Effectiveness of HAR has not been quantified at this time. The user can check on the progress of current research to determine the adequacy of the given estimated value.

$$\text{DELAY}\Big|_{W/CTL} = \int_0^{T_{CC2}} N_{DC}(t)dt - \int_0^{T_{CC2}} N_C(t)dt \tag{26}$$

$$= NDI_2 - NCI_2$$

and the delay benefit for the single occurrence of an incident is:

$$\text{VDB} = \text{DELAY}\Big|_{W/O\ CTL} - \text{DELAY}\Big|_{W/CTL} \tag{27}$$

Table 48 provides the analytical equations required to develop the incident benefit. In order to use these equations effectively it is very important to recognize the meaning of the input flow (Q1) and time interval (TC_1). Q_1, Incident Capacity, is defined for a typical incident where one lane is blocked. Since freeways typically have from 2 to 5 lanes, Q1 is freeway dependent. Figure 28 provides an estimate of the capacity remaining with one lane blocked as a function of the number of freeway lanes. The parameter TC_2, the time lag until control is implemented, is principally a function of the surveillance detector spacing. Figure 29 presents the consensus of several research efforts.

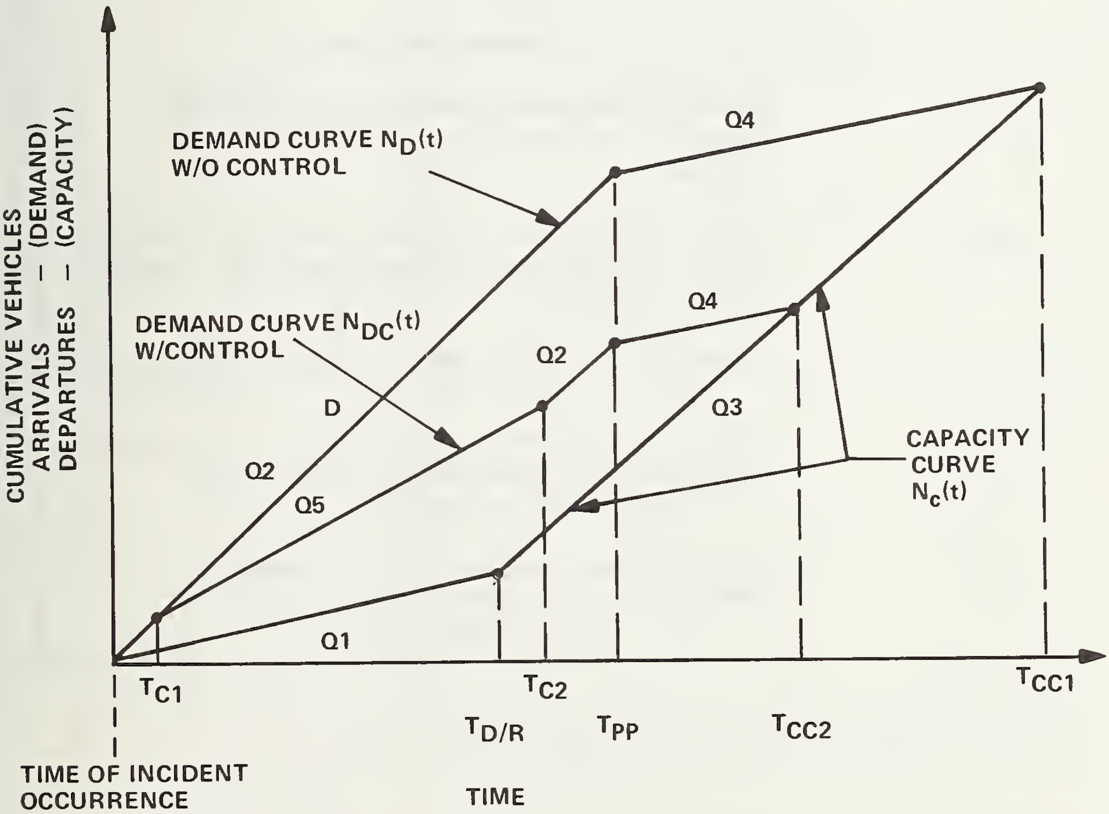

Figure 27. Incident Delay Diagram

Table 47. Definition of Variables for Incident Delay in Figure 27

- Demand and Capacity Flow Levels*

1. Q_1	Capacity with Incident on Roadway. Typical value = (γ) (Q_3). See Figure 28 for γ.
2. Q_2	Expected Peak Period Demand (average over peak period). Derived from Data Base. Typical value = (.95) (Q_3).
3. Q_3	Normal Capacity of Roadway derived from Data Base. Typical value = 1800 VPLPH.
4. Q_4	Expected off-peak period demand (average over period). Derived from data base. Typical value = (.7) (Q_3).
5. Q_5	Expected Demand at Incident Site with Control Implemented. $Q_5 = Q_2 - \Delta Q$

- Time Intervals (Relative to Time of Incident Occurrence)

1. $T_{D/R}$	Expected Duration of Incidents on Roadway. Derived from Data Base. Typical Value = 3.5 MIN.
2. T_{C1}	Time Interval after Incident until Control Implemented. Function of Detector Spacing. Typical Value = 8 MIN.
3. T_{C2}	Control Termination Time. Value = $(T_{D/R} + T_{C1})$
4. T_{PP}	Expected Termination Time of Peak Period given an Incident has occurred. Typical Value = (.5) (Peak Period Interval)
5. T_{CCi}	Time at which Incident Caused Congestion is eliminated (=1 - No Control) (=2 - With Control)

*All flow levels in units vehicles per lane per hour.

Table 48. Incident Benefit Equations (Reference Figure 27)

- Congestion Termination Time - T_{CCi}

W/O Control - $$T_{CC1} = (Q_3-Q_4)^{-1} [(Q_2-Q_4)\, T_{PP} + (Q_3-Q_1)\, T_{D/R}] \tag{28}$$

W/ Control - $$T_{CC2} = T_{CC1} - (Q_2-Q_5)\,(Q_3-Q_4)^{-1}\,(T_{C2} - T_{C1}) \tag{29}$$

- Demand and Capacity Equations*

$$NCI_i = \frac{(Q_3-Q_1)}{2}\, T_{D/R}^2 + \frac{Q_3}{2}\, T_{CCi}^2 - (Q_3-Q_1)\, T_{D/R}\, T_{CCi} \tag{30}$$

$$NDI_1 = \frac{Q_4}{2}\, T_{CC1}^2 + (Q_2-Q_4)\, T_{PP}\, T_{CC1} - \frac{(Q_2-Q_4)}{2}\, T_{PP}^2 \tag{31}$$

$$\begin{aligned} NDI_2 = \frac{Q_4}{2}\,(T_{CC2}-T_{PP})^2 &+ \frac{Q_2}{2}\,[T_{C1}^2 + 2\,T_{C1}\,(T_{CC2} - T_{C1})] \\ &+ \frac{Q_2}{2}\,[(T_{PP}-T_{C2})^2 + 2\,(T_{PP}-T_{C2})\,(T_{CC2}-T_{PP})] \\ &+ \frac{Q_5}{2}\,[(T_{C2}-T_{C1})^2 + 2\,(T_{C2}-T_{C1})\,(T_{CC2}-T_{C2})] \end{aligned} \tag{32}$$

*If $T_{CC2} < T_{PP}$ and $T_{CC1} > T_{PP}$ Then set $T_{PP} = T_{CC2}$ in NDI_2 equation

If $T_{CC2} < T_{PP}$ and $T_{CC1} < T_{PP}$ Then set $Q_4 = Q_2$ in NDI_1 equation

and set $Q_4 = Q_2$, $T_{PP} = T_{CC2}$ in NDI_2 equation

The output of these equations provides the peak period delay benefit per incident. In a sequence of steps similar to those given in Table 48, this per-incident benefit is expanded to a total benefit for the entire corridor. The sequence of steps is:

(1) Establish the level of control (ΔQ) which the maximum performance system can shift between the corridor roadways.

(2) Compute the benefit for the occurrence of a single incident. The benefit is defined for a user-specified set of incident parameters (Table 47).

(3) Compute the total yearly incidents for limited access roadways of the corridor. Total lane blocking incidents is obtained by multiplying the peak period rate (from Chapter 7) by lane-miles of roadway and by expansion factors given in Table 49.

(4) Multiply yearly incidents by the benefit for a single incident and the control probability factor (Table 32) for each configuration set of corridor roadways.

(5) Sum the yearly incident benefits developed for each configuration in step (4). This final value is the delay benefit derived from yearly operation of the most versatile system during peak travel periods.

Figure 30 provides a block diagram of the complete incident benefit computational procedure. For each candidate system the reduction in performance with respect to the most versatile system is handled in a procedure identical to that given in the last section.

15.5 OFF-PEAK INCIDENT AND MAINTENANCE BENEFIT

This benefit category addresses the requirement of IMIS to maintain the quality of flow and roadway performance during non-peak time periods. The control concept and methodology for obtaining benefits is completely analogous to procedures given in the previous section with changes required only for off-peak flow conditions.

Benefits are assumed to be developed in the off-peak periods only from the occurrence of accident/incidents or when maintenance operations require the partial closure of a roadway. Normal congestion is not included since demand is usually sufficiently below nominal capacity.

In general, the differences between the off-peak benefit procedure and the peak benefit procedure are those of degree. The differences are principcally related to the following factors:

(1) The traffic demand is substantially below roadway capacity hence the level of congestion and the benefits per incident due to system operation are substantially reduced.

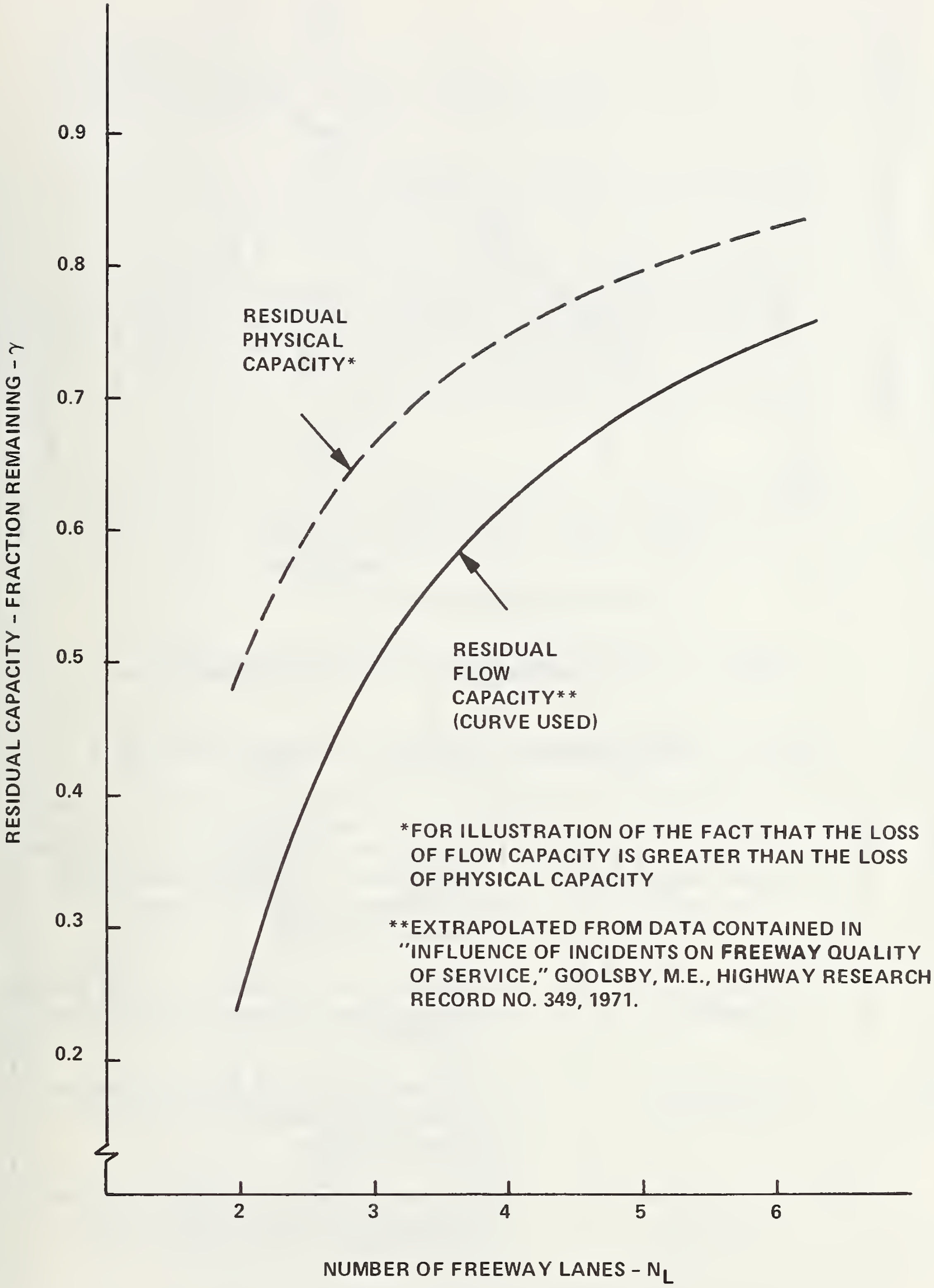

Figure 28. Residual Capacity with One Lane Blocked

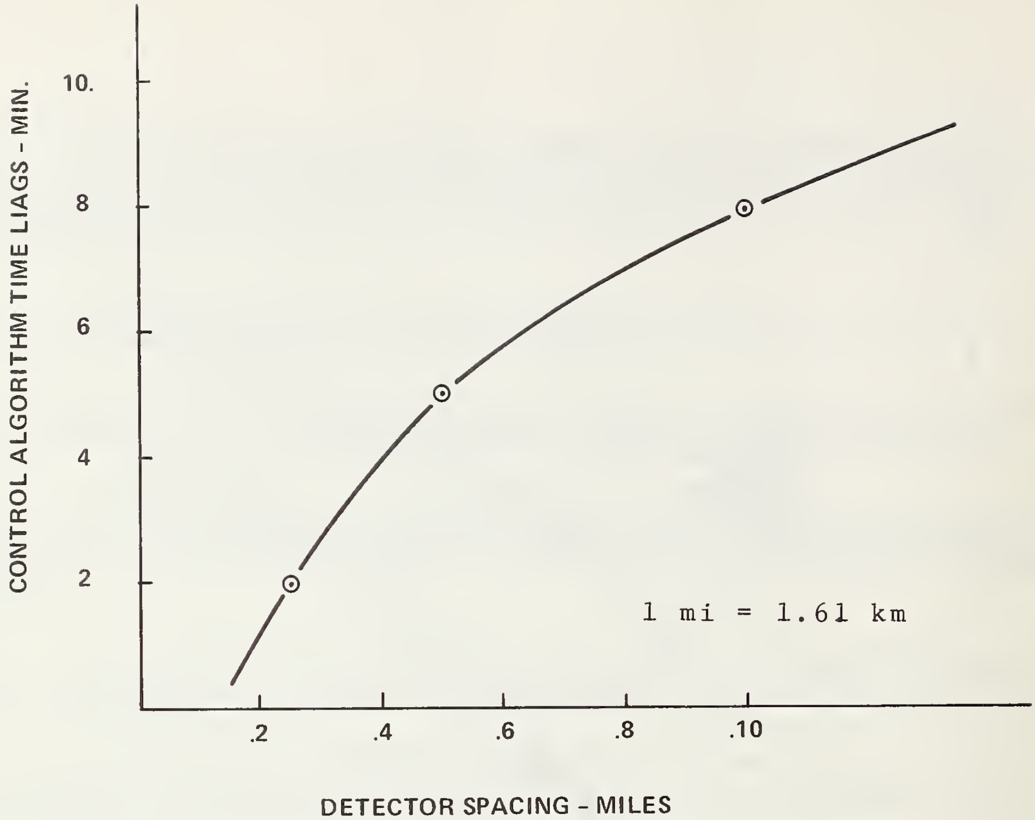

Figure 29. Typical Time Lag for Incident Detection

Table 49. Benefit Expansion Factors, Peak Period Incident

Factors	Description
K1 - Peak Period Duration	Duration of a weekday peak period. This factor should be estimated for each corridor.
K2 - Peak Period Number	Number of Peak Periods per day during which system is used. Typical value = 2 per day.
K3 - Number of Days/Year	Number of days in a year considered for benefit computation. Typical value = 260 days.
K4 - Peak Period Direction	Number of traffic flow directions to be considered during a time period. Typical value = 1 direction
K5 - Number of Limited Access Roadways per Configuration	Typical values: Configuration Type 1, 3 = 2 Configuration Type 2 = 1

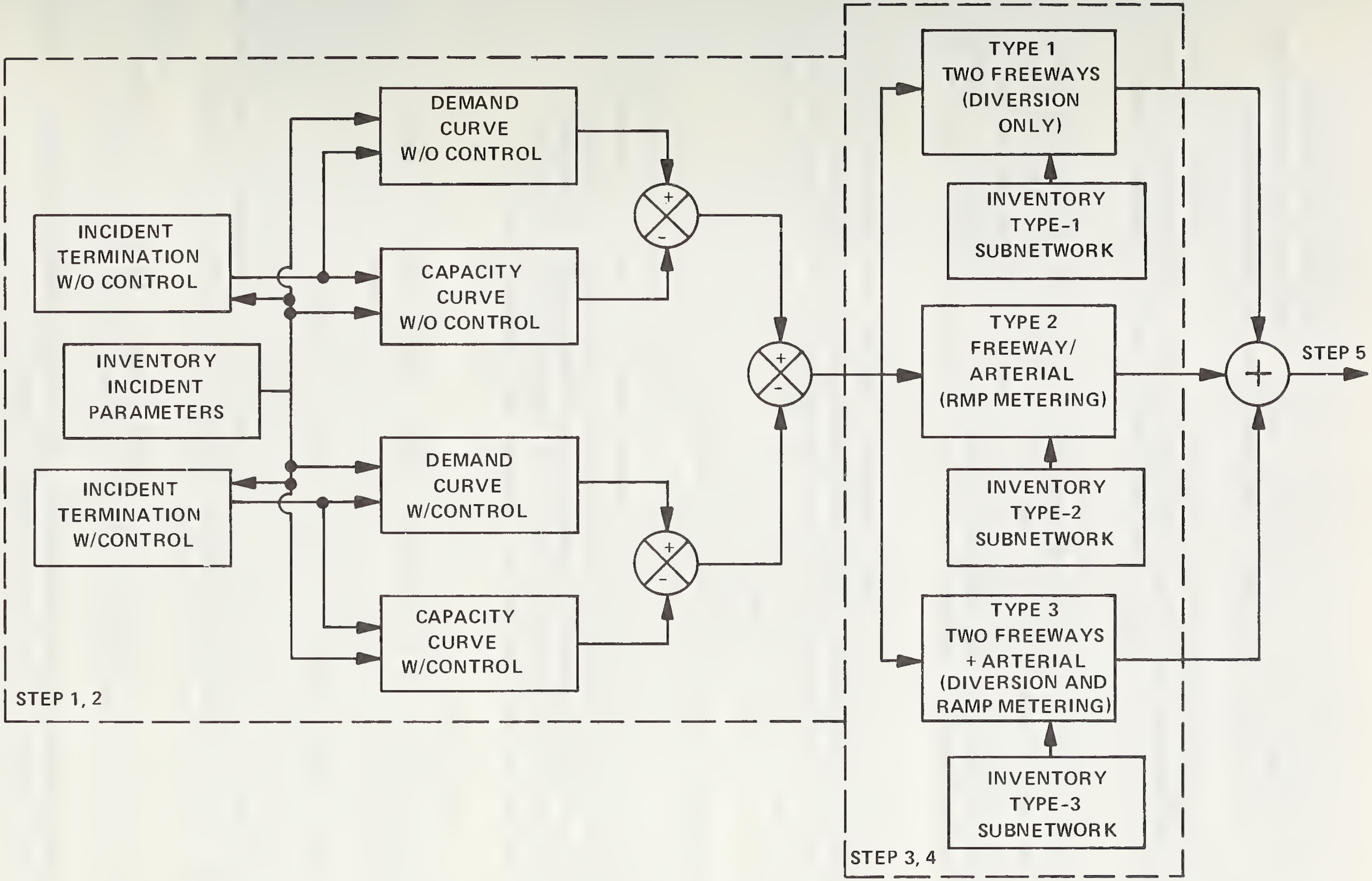

Figure 30. Computational Procedures, Peak Period Incident Benefit

(2) The probability that control can be exercised after the occurrence of an incident is assumed to be 1 for all roadway types. (The lower demand volume implies there is always excess capacity so that traffic can be shifted between roadways.)

(3) While the volumes are lower, the longer duration of the off-peak time period implies that in general a higher total of accidents and incidents will occur over a year.

The fundamental benefit level for the off-peak period can be obtained with the same equation set utilized for peak periods with the substitution of Q4 for Q2. Lowering the demand level dramatically reduces the level of benefit that can be obtained for a single incident. The procedure for expansion of benefits for a yearly level is also identical with the substitution of a revised set of expansion factors (Table 50).

A secondary benefit which can be related to the off-peak incident benefit is that associated with reducing the adverse effects of lane blocking maintenance operations. These operations generally occur during the mid-day off peak-period. They generally occur over the entire period on the average of once a week on each major limited access corridor roadway.

15.6 ACCIDENT REDUCTION AND DELAY BENEFITS

The previous paragraphs developed yearly benefits based exclusively on the operational performance parameter of vehicle hours of delay. The benefit associated with accident reduction is composed of two parts -- a reduction in the yearly accident costs (fatal, non-fatal and property damage) and a reduction in the vehicle hours of delay associated with the accidents which did not occur. IMIS reduces the rate at which accidents occur for the following reasons:

- flow instabilities are minimized
- communication to motorists of downstream flow disturbances is improved
- shift of motorists away from problem areas is improved.

The calculation of the accident reduction benefit is performed in two parts corresponding to each benefit component. The reduction predominantly occurs during the peak travel periods when the application of system-wide control policies will produce the greatest improvement in quality of flow.

The procedure for developing the accident reduction benefit is composed of two factors:

- Determination of an historical accident rate for the limited access roadways of the corridor. This is an output of Chapter 7.

Table 50. Benefit Expansion Factors, Off Peak Period Incident

Factors	Description
K1 - Off Peak Period Duration	Duration of a weekday off-peak period. This factor should be estimated for each corridor.
K2 - Off-Peak Period Number	Number of off-peak periods per day during which system is used. Typical value = 1 per day.
K3 - Number of Days/Year	Number of days in a year, considered for benefit computation. Typical value = 260 days
K4 - Off-Peak Period Direction	Number of traffic flow directions to be considered during a time period. Typical value = 2 directions.
K5 - Number of Limited Access Roadways per Configuration	Typical values: Configuration Type 1, 3 = 2 Configuration Type 2 = 1

- Specification of an accident reduction factor correlated to system operation. Based on previous evaluation studies of operational systems an expected reduction in accidents on the order of 17 to 26 percent is achievable. An average value of 21.5 percent can be used as representative of these reductions.

Multiplication of these two terms by the number of lane-miles of limited access roadway on which ramp metering is available (subnetwork types 2 and 3) and by the benefit expansion factors contained in Table 49 results in a yearly accident reduction for the corridor. The relationship of this benefit to the spectrum of alternate system designs is achieved through the reduction of ramp metering locations on a lane-mile basis using the maximum system design as the baseline. Thus the benefit relationship is:

$$ARBY = .215\ (ACC)\ K_1\ K_2\ K_3\ K_4\ (K_5\ LM_2 + K_5\ LM_3) \qquad (33)$$

where

ARBY - accident reduction benefit - accident/year

ACC - historical accident rate, limited access, corridor/or roadways

K_i - expansion factors, Table 49 ($i = 1, \ldots . 5$)

LM_j - lane-miles of limited access corridor roadway for each system design corresponding to configuration type j.

The second part of this benefit category is the delay savings due to the occurrence of a reduced number of accidents. The concept of this benefit differs from the previously derived delay benefits (Sections 14.3, 14.4 and 14.5) in that those benefits accrue to IMIS after the incident has occurred and the system is attempting to minimize the extent of the resulting congestion. This delay benefit category accrues to IMIS based on the fact that delay associated with an accident which does not occur (with respect to the prior historical corridor rate) is a benefit for the system.

The methodology used to calculate this benefit is a multiplication of the accidents saved (ARBY) with the delay saved per peak period accident.

Referring to the Incident Delay Diagram (Figure 27) the total delay saved is recognized as the area between the demand curve without control and the capacity curve. However, since the peak period incident benefit (Section 14.4) has already taken a delay benefit based on the historical corridor accident rate, the net additional benefit is the area between the demand curve with control and the capacity curve. Thus the procedure for calculating this delay benefit is functionally related to the peak period incident benefit computation.

Using the terminology and equations given in Tables 47 and 48, the standard delay benefit due to a reduction of one accident is:

$$VDB_A = NDI_2 - NCI_2 \tag{34}$$

Therefore the total additional yearly delay benefit due to the reduction in the accident rate

$$VDBY_A = (VDB_A)\ (ARBY) \tag{35}$$

15.7 FUEL AND POLLUTION BENEFIT COMPUTATION

The improvements in traffic flow achieved as a result of IMIS controls (e.g., decreases in number of stops, idle time, acceleration and decelerations) results in concomitant reductions in fuel consumption and pollutant emissions. During the Long Island IMIS feasibility study, a model was developed to quantify these benefits. (The model development is described in the IMIS Phase I Final Report referenced earlier, i.e., document FHWA-RD-77-47.)

The objective of the model was to relate both fuel consumption and pollution directly to vehicle delay since the latter is the major parameter used in IMIS benefit evaluations. The results of the study, which included computer simulation runs, indicated that a linear model could be used for the desired relationships. Specifically, the model equations are as follows:

$$FBY = .96 \text{ VDBY} \quad (36)$$

$$PBY_{CO} = 2.1 \text{ VDBY} \quad (37)$$

$$PBY_{H} = 0.1 \text{ VDBY} \quad (38)$$

where FBY = Yearly fuel benefit in gallons saved (1 gallon = 3.8 liters)

PBY_{CO} = Yearly pollution benefit, carbon monoxide, in pounds eliminated (1 pound = .45 kilograms)

PBY_{H} = Yearly pollution benefit, hydrocarbons, in pounds eliminated (1 pound = .45 kilograms)

$VDBY$ = Total yearly vehicle delay benefit, in vehicle-hours (sum of all components)

Thus, once the total vehicle delay benefit has been computed for each alternative design, the corresponding fuel and pollution benefits may be estimated using the above relationships.

15.8 YEARLY BENEFIT FOR EACH CANDIDATE SYSTEM

As each category of benefit is computed for each candidate system, the component is recorded on a system benefit work sheet. Table 51 illustrates the framework of this worksheet. The individual benefit categories for vehicle delay, accidents, fuel and pollutants are shown for each candidate system. As each category of benefit is calculated for each system the worksheet is filled in.

Table 51. Typical System Benefit Worksheet

BENEFIT CATEGORIES	CANDIDATE SYSTEM DESIGNS								
	D1	D2	D3	E1	E2	E3	F1	F2	F3
A. Delay Saved (VDBY in thousands of vehicle-hrs/yr)									
(1) Peak Period									
• Normal	1417	1327	984	1393	1304	972	1161	1056	475
• Incident	766	727	582	757	719	454	477	439	205
(2) Off Peak									
• Incident	508	483	386	502	477	301	491	452	212
(3) Accident Reduction	110	110	110	110	110	87	143	143	113
(4) Total	2801	2647	2062	2762	2610	1814	2272	2090	1005
B. Fuel Saved (FBY, in thousands of gallons/yr)	2689	2541	1980	2651	2506	1741	2181	2006	965
C. Accident Reduction (ARBY, in # of accidents/yr)	200	200	200	200	200	158	200	200	158
D. Pollution Reduction (PBY, in thousands of pounds/yr)									
CO	5882	5559	4330	5800	5481	3809	4771	4389	2110
HC	280	265	206	276	251	181	227	209	100

Note: 1 pound = .45 kilograms; 1 gallon = 3.8 liters

CHAPTER 16

BENEFIT/COST EVALUATION OF ALTERNATIVE SYSTEMS

16.1 INTRODUCTION

16.1.1 Objectives

- To provide a basis for comparing the alternative designs with respect to their projected benefits and costs.
- To provide guidance with respect to the comparison of benefits and costs between the several alternative systems.
- To provide a procedure for assessing the sensitivity of the evaluation to uncertainty in methodology parameters.
- To provide guidance with respect to the selection of one of the alternative systems.

16.1.2 Inputs

- The specifications of the alternative system designs as developed in Chapter 13.
- The costs of each alternative design as developed in Chapter 14.
- The benefits for each alternative design as developed in Chapter 15.
- Various models and relationships developed earlier.

16.1.3 Outputs

- A tabular and graphical presentation of benefit/cost data for the alternative designs.
- The benefit/cost variations due to parameter uncertainty.
- Analysis of alternative designs based on benefit/cost consideration.

16.2 DOLLAR BENEFIT AND COST DATA

In Chapter 15 the several categories of benefits for each alternative design were developed with respect to vehicle delay, fuel consumption and pollution. Before a comparison of benefits and costs can be made, the benefits must be converted to a dollar equivalent. The equation for transforming benefits to a dollar equivalent is:

$$\text{Dollar Equivalent} = (\$VD)\,(VDBY) + (\$F)\,(FBY) + (\$AC)\,(ARBY) \qquad (39)$$

where

$VD - dollar conversion factor for vehicle-hour of delay *

$F - dollar conversion factor for gallon of fuel (gasoline)*

$AC - dollar conversion factor for cost of weighted composite traffic accident *

The benefit quantities VDBY, FBY, and ARBY were computed for each alternative design in Chapter 15 and were summarized in the system benefit worksheet. Equivalent annual costs were developed and summarized in Chapter 14.

The dollar benefits and costs for each alternative design are now tabulated and plotted. As a typical example, data for six hypothetical alternative designs are shown in Table 52, along with their calculated benefit/cost ratios. Figure 31 illustrates the graphical format. The ordinate represents the annual benefit, while the abscissa represents the equivalent annual cost. The slope of the line connecting each point (system) to the origin is then the benefit/cost ratio.

16.3 EVALUATION OF ALTERNATIVE SYSTEMS **

Two evaluation criteria, both based on benefit and cost data, are suggested for use in the selection of one of the alternative systems. The first is the basic benefit/cost ratio, formed by simply dividing the annual benefits by the equivalent annual costs. This is a direct measure of the dollars returned for each dollar invested, and the "best" system is the one with the highest ratio. Any system whose ratio is less than one is discarded from further consideration since it represents a net loss for the investment. For the hypothetical systems shown earlier, systems B1 and C1 should be discarded on this basis. System C2, having the highest benefit/cost ratio, is considered best based on the maximum benefit/cost criterion.

The second evaluation criterion is the "incremental" benefit/cost ratio. This criterion assesses the relative worth of additional investment, i.e., whether the additional dollars invested will produce more than a one-for-one return in benefits. The procedure used for calculating the incremental benefit/cost ratios is to list the systems (except those excluded because their B/C is less than 1.0) in order of increasing cost, along with their associated benefit and cost data. The lowest cost system serves as the initial baseline. The incremental cost and incremental benefit of the next system, relative to the baseline, is computed and an incremental benefit/cost ratio is formed. If the incremental benefit/cost ratio is greater than 1.0, this system becomes the new baseline; otherwise the original baseline remains. The process is

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* The selection of these conversion factors should be based on the standard values used by the State DOT for evaluation of similar traffic and safety improvements and systems.

** The evaluations discussed below can also be done graphically using the slopes of lines on the benefit versus cost graph.

Table 52. Hypothetical Benefit and Cost Data

Alternative System Designation	Equivalent Annual Cost*	Annual Benefits*	Benefit/Cost Ratio
A1	2.75	4.00	1.45
A2	4.30	4.78	1.11
B1	6.48	4.78	0.74
B2	3.28	6.55	2.00
C1	3.52	2.53	0.72
C2	2.25	5.00	2.22

* In millions of dollars

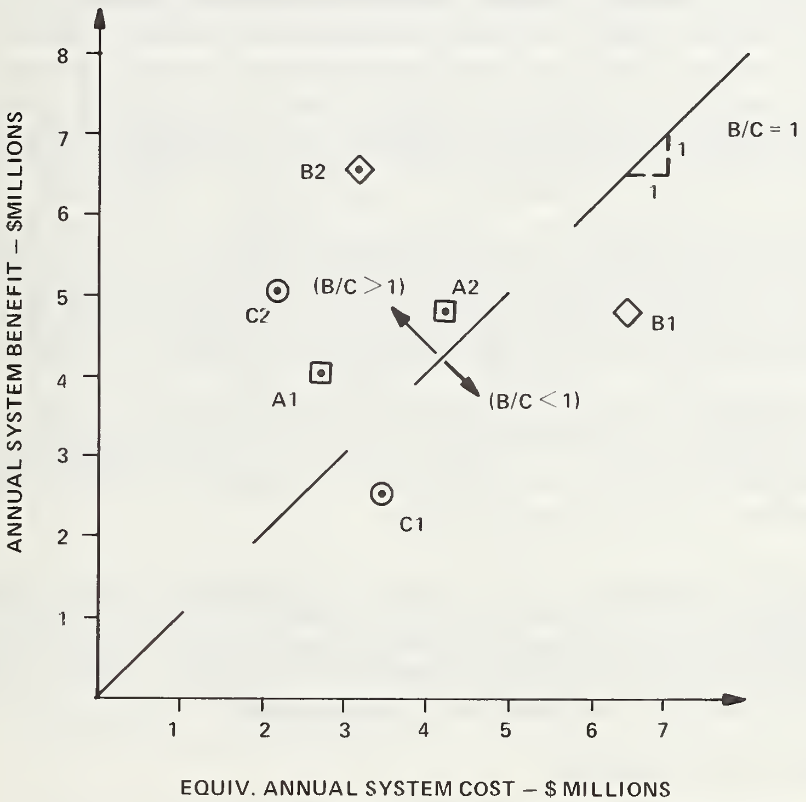

Figure 31. Typical Plot of Benefit and Cost Data

continued until all systems have been considered, changing the baseline as indicated. The final baseline system is considered "best" based on the incremental benefit/cost criterion. A sample calculation is shown below for the four remaining hypothetical systems to illustrate the procedure.

System	Cost	Benefit	ΔCost	Benefit	Incr. B/C	
C2	2.25	5.00	-	-	-	(Initial Baseline)
A1	2.75	4.00	0.50	-1.00	-2.00	
B2	3.28	6.55	1.03	1.55	1.50	(New Baseline)
A2	4.30	4.78	1.02	-0.22	-0.22	

It is seen from the above that System B2 would be the "best" system on the incremental benefit/cost evaluation criterion.

In the foregoing hypothetical case, each evaluation criterion produced a different "best" system. These results, however, are not conflicting; rather they represent two different philosophies. The first seeks to maximize the return on the investment, while the second considers additional investment to be warranted as long as the dollar return continues to exceed the dollar invested (at least up to a given cost constraint). Neither philosophy should be considered totally correct in and of itself. For example, suppose that in an IMIS corridor the metering of one or two ramps could alleviate a congestion problem on one of the freeways. This "system" would undoubtedly have an extremely high benefit/cost ratio. However, it only addresses one small portion of the problems in the corridor. Are not the other problems worthy of action even though they will reduce the benefit/cost ratio? And carried to the other extreme, should millions of dollars continue to be invested, perhaps at the expense of other projects, simply because they are returning more than a dollar for each one invested?

These examples, though extreme, serve to indicate the potential problem of relying solely on either approach. Therefore, both should be considered as inputs to the system selection process, along with due consideration of the corridor needs, budgetary constraints, and of course judgment.

Another important factor should be kept in mind when comparing the alternative system designs. This is, that for certain system elements, a cost has been included but benefits have not. Generally, the related benefits have not been included because of the difficulty in establishing their quantitive values. Thus, benefits for the following are not included in the analysis:

- Motorist Aid Callboxes
- Added Patrolling Vehicles (Police cars and/or Tow Trucks)
- Pre-trip Information Services

- Full Time (as opposed to half-time) staffing of the Control Center
- Use of Dual Computer configuration (versus single computer configuration) at the Control Center.

Some allowance for this should be made in the design comparisons. For example, a more versatile system might be selected over a "less versatile" candidate even if the former has a somewhat lower benefit/cost ratio, if the user considers the non-quantified benefit items to be of substantial importance to the system.

16.4 BENEFIT/COST SENSITIVITY ANALYSIS

The specific benefits, costs and ratios developed for each alternative system are in fact estimates of the 'true' system values and thus are subject to variations caused by uncertainties in assumptions about individual components used in the computation. In this section, procedures are given to identify the limits of these benefit/cost variations. To determine these probable limits, three steps must be completed. These are: identify the basic factors which have a pronounced effect on the benefit/cost relationships, determine the probable range of variation for their associated quantities and assess the probable effect on these variations on the benefit/cost system values.

Table 53 identifies the set of factors which have a major impact on benefits and cost variations. The first eight of these affect system benefits, while the last three affect system costs.

The procedure is to treat each group separately and subsequently combine the uncertainties to arrive at the overall variations. Basically, an error analysis approach can be used to relate the total benefit or cost error to the errors in their component quantities. Under the assumption that the factors are independent, the total benefit or cost sensitivity is obtained by taking the square root or the sums of the squares of the deviations caused by the individual variations due to each component. This general procedure is applied to the benefit and cost relationships respectively in the following paragraphs.

16.4.1 Benefit Sensitivity Analysis

The yearly system benefit, in dollars, is defined by the following equation:

$$\$B = \$VD\left[(KI_p)(VDB_{Ip}) + (KI_{op})(VDB_{Iop}) + VDBY_c + (ARBY)(VDB_A)\right] + (\$F)(FBY) + (\$AC)(ARBY) \tag{40}$$

where:

\$B	- Yearly system benefit (dollars)
VDB_{Ip}	- Delay saved/peak period incident - 1*, 2, 3
VDB_{Iop}	- Delay saved/off-peak period incident - 1, 2, 3
KI_p	- Yearly total lane blocking incidents, peak period -6
KI_{op}	- Yearly total lane blocking incidents, off-peak period -4
$VDBY_c$	- Annual delay saved, normal congestion
\$VD	- Benefit/vehicle hour delay saved -4
\$AC	- Benefit/accident saved -5
FBY	- Yearly total fuel saved -7
\$F	- Benefit/gallon fuel saved -8
ARBY	- Yearly reduction peak period accidents -6
VDB_A	- Delay saved/accident eliminated

The nominal values for the individual quantities have been established in the earlier benefit analyses, and should be the ones used in the sensitivity analysis.

The computation of benefit sensitivity is affected by the relationship of each component to the benefit, and the expected variation of each component. Each of these relationships can ultimately be put in the form:

$$\Delta B_x = a_x \quad (\Delta X) \tag{41}$$

Utilizing this general relationship, the variation in each component (ΔX) makes a distinct contribution (ΔB_x) to the overall benefit uncertainty. The "sensitivity coefficient" for each component (a_x) is unique for each component. Table 54 gives the sensitivity coefficients for each of the components given in Equation 40.

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* The numbers which follow the definitions are cross-references to Table 53.

Table 53. Benefit/Cost Sensitivity Factors

•	Benefit Factors
(1)	Motorist Response to Advisory Signing
(2)	Average Incident Detection Time
(3)	Average Time to Clear Incident
(4)	Value of Motorist's Time (dollar)
(5)	Cost of Accidents (dollar)
(6)	Accident Frequency
(7)	Fuel Saved
(8)	Value of Fuel (dollar)
•	Cost Factors
(9)	Interest Rate
(10)	Useful Life
(11)	Component System Costs

The following paragraphs include a further discussion of the benefit factors of Table 53, and provide guidance for determining variations in related quantities. Unless otherwise noted, a variation of ± 10 percent may be assumed if more specific values cannot be determined.

Motorist response to advisory signing influences system benefits through the control volume shift (ΔQ). Variations in ΔQ impact the delay-saved benefits of peak-period normal congestion and incident congestion for both peak and off-peak periods.

Incident detection time and incident clearance time only affect incident congestion. The incident detection time is primarily a function of detector spacing. For a detector spacing of 0.5 miles (0.8 km) the nominal detection time is 5 minutes while for 1.0 mile (1.6 km) spacing, it is 8 minutes. Incident clearance time is composed of three distinct components, incident response, incident removal, and, for accidents, investigation times.

The value of a motorist's time is used to convert all vehicle-hours of delay saved to a dollar benefit. In general, each state has a standard value* for motorist time or a vehicle-hour of delay which is periodically updated to reflect current

Table 54. Sensitivity Coefficients

Component (ΔX)	Coefficients (a_x)**
VDB_{Ip}	$\$VD\ (KI_p)$
VDB_{Iop}	$\$VD\ (KI_{op})$
KI_p	$\$VD\ (VDB_{Ip})$
KI_{op}	$\$VD\ (VDB_{Iop})$
$\$VD$	$(KI_p \cdot VDB_{Ip} + KI_{op} \cdot VDB_{Iop} + VDBY_c)$
	$+ ARBY \cdot VDB_A$
$\$AC$	ARBY
FBY	$\$F$
ARBY	$\$VD\ (VDB_A) + \$AC$

motorist costs. It is important to keep in mind that the benefits generated are in the form of vehicle-hours of delay saved. Thus, if the value available is in units of motorist's time, this must be multiplied by the average vehicle occupancy to convert it to vehicle-hour units.

Cost of accidents is a conversion factor which is a weighted average of accident costs for fatal, injury, and property damage only categories. The assignment of a numerical value should be based on state-wide statistics. State DOT's maintain standard factors, periodically revised, to use for evaluation programs.

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* For example, NYSDOT as of April 1976 used a value of \$3.42 per vehicle-hr of delay. This factor takes into account the vehicle occupancy for NYS of 1.3 passengers/vehicle

**The coefficients for each component are determined by inspection of Equation 40.

Accident frequency on corridor freeways can have a high degree of variability. The assignment of a nominal frequency was discussed in Chapters 6 and 7. Variability of accident frequency is caused by two factors, the variability of year to year accident statistics and the error in the estimate of accident reduction resulting from system operation. These factors are independent and hence can be separately treated.

The variation in accident frequency can be determined from state statistics which would typically show a variation of 3 to 4 percent from year to year. The variation in accident reduction is determined by collecting data from before/after studies from surveillance and control systems which have been implemented. As noted in Chapter 15, typical accident reductions on the order of 20 percent would be expected after system implementation. The normal range of reduction is from 17 to 26 percent.

The variation in fuel saved is obtained as a direct relationship with the variation in vehicle-hours of delay saved. Therefore, the same coefficient (.96) is used to obtain the fuel saved variation.

The variation in fuel cost is a function of brand, octane rating, purchase location, and company marketing/pricing policies. The combined effect of these factors may be accounted for with an overall variation of ±10 percent or ±6 cents per gallon (±1.6 cents/liter).

At this point, the variation in system benefit (ΔB_X) for each component error source (ΔX) can be obtained utilizing the sensitivity coefficients (A_X) given in Table 54. The total expected variation in system benefit is then computed as the square root of the sum of squares of the individual variations which are independent. Thus:

$$\Delta B = \left[\sum_X (\Delta B_X)^2 \right]^{1/2} \qquad (42)$$

16.4.2 Cost Sensitivity Analysis

Errors in system cost are sensitive to variations in interest rates, useful life and component costs. Careful consideration should be given to each of these parameters including their nominal values used in the cost computation.

Interest rates and useful life were discussed in Chapter 14. The nominal value for interest rate proposed as a reasonable value in today's fiscal environment is 10 percent. The variation about this value could be on the order of ± 20 percent which indicates a range of from 8 to 12 percent. Of course values should be used which reflect current fiscal policy of the state in which the corridor is located.

Useful life for electronic equipment and associated control and surveillance hardware has a nominal value of 15 years. The variation about this value is on the order of ±5 years.

Typical cost estimate variations should be set at ±20 percent about the estimated capital, maintenance and operational cost values developed from the procedures given in Chapter 14. This range should be modified, if necessary, based on applicable cost history in the local geographical area.

The approach used to compute the overall system cost variation is based on computing the individual cost variations of each cost factor. The range of each system cost component is obtained by holding each cost factor except one to its nominal value. The square root of the sums of the squares of the resulting differences between these values and their corresponding nominal values are then calculated to provide an estimate of overall cost variation.

The computation requires a conversion of the capital cost estimates to equivalent annual values, using the capital recovery factors (CRF) applicable to each interest rate and useful life cycle. Table 55 shows the results for a typical computation. Baseline values shown in the table are based on nominal values of 10 percent interest rate, and 15 years useful life, with other cost estimates being kept at their nominal values. The equivalent annual capital costs are added to the maintenance and operational costs for each of the cost element variations, and these totals are subtracted from the baseline total to give the individual cost element differences.

To obtain the overall cost sensitivity, the cost differences for each category are averaged, and the square root of the sums of their squares is computed, giving the error ΔC.

16.4.3 Overall Benefit/Cost Ratio Sensitivity

The previous sections discussed the development of the individual benefit and cost error relationships. The benefit/cost ratio error relationship is:

$$\Delta\left(\frac{B}{C}\right) = \left[\left(\frac{\Delta B}{C}\right)^2 + \left(\frac{B}{C}\right)^2\left(\frac{\Delta C}{C}\right)^2\right]^{1/2}$$

where ΔB and ΔC are the errors and B and C are the nominal benefit and cost generated for a specific alternative system.

Table 55. Sample Computation Of Cost Element Variations

Factor	Baseline Value*	Interest Rate		Useful Life		Cost Estimate	
		8%	12%	(20 yrs)	(10 yrs)	(-20%	+20%)
Capital Recovery Factor (CRF)	0.1315	0.1168	0.1468	0.1175	0.1628	-	-
Annual Cost of Capital (M$)	4.058	3.604	4.530	3.626	5.024	3.246	4.870
Ann. Maint. $ Oper. Cost (M$)	1.535	1.535	1.535	1.535	1.535	1.228	1.842
Total Annual Cost (M$)	5.593	5.139	6.065	5.161	6.559	4.474	6.712
Cost Differences (M$)	-	0.454	0.472	0.432	0.966	1.119	1.119

* Based on 10% interest rate and 15 year useful life

APPENDIX A

ORIGIN-DESTINATION PATTERN ESTIMATION

A.1 OBJECTIVE

To develop an estimate of the origin-destination pattern for the major limited access facility or facilities in the corridor. The estimate is made for a typical peak hour. The results will be used subsequently to develop average trip length, assess diversion potential, and estimate benefit reduction associated with fewer diversion points.

A.2 INPUTS REQUIRED

(1) A sequential listing (upstream to downstream) of all entrance and exit ramps in the corridor for each major limited access facility.

(2) A balanced hourly volume network for each facility.

A.3 OUTPUT

A completed worksheet for each facility. For each entrance ramp, the worksheet will show how its volume distributes to all downstream exit ramps. For each exit ramp, the worksheet will show the component volumes coming from each upstream entrance ramp.

A.4 TERMINOLOGY

For convenience, all ramps are assigned numbers, with odd numbers being used for entrance ramps and even numbers used for exit ramps. The mainline input at the entrance to the corridor is considered as the first entrance ramp. The mainline output at the end of the corridor is considered as the last exit ramp. The following symbols are then used:

i = entrance ramp. $i = 1,3,5,7....$
j = exit ramp. $j = 2,4,6,8....$
Ri = total volume entering at entrance ramp i, (vehicles per hour-vph)
Rj = total volume exiting at exit ramp j, (vph)
Rji = the volume component of Rj which entered at Ri, (vph)

Example: if 450 vph exit at ramp 6, and 103 of these originated at entrance ramp 3, then R6 = 450 vph and R63 = 103 vph.

It follows that $\sum_i Rji = Rj$ for all entrance ramps i upstream of exit ramp j.

k = mainline location between two adjacent ramps. Use number of upstream ramp for k.

Mk = mainline volume at point k, (vph)

Mki = the volume component of Mk that originated at entrance ramp i, or in other words, the amount of the original entrance volume from i that is still on the mainline at k (i.e., has not yet exited), (vph).

Example: If the mainline volume between ramps 5 and 6 is 4900 vph, and 1122 vph of these came from entrance ramp 3, then M5 = 4900 vph and M53 = 1122 vph.

It follows that $\sum_i Mki = Mk$ for all entrance ramps i upstream of mainline location k.

fj = exit fraction for exit ramp j, i.e., exit ramp volume divided by the mainline volume immediately upstream.

Example: If R6 = 480 vph and M5 = 4900, then

$$F6 = \frac{R6}{M5} = \frac{450}{4900} = .092$$

A.5 PROCEDURE

Step 1. Assign ramp numbers to the sequential ramp listing.

Step 2. Set-up a work sheet. Lay out a single-line diagram of the limited access facility showing all ramps. Add ramp and mainline volumes and ramp numbers. Each calculation result can be entered on the work sheet.

Step 3. Starting at upstream end, calculate the first exit fraction.

Step 4. Calculate the corresponding set of Rji.
$Rji = fj \times Mki$

Step 5. Subtract each Rji from its corresponding Mki to obtain the next downstream set of Mki.

Step 6. When the next ramp is an entrance ramp, the previous Mki are simply repeated and the value of the entering volume is added to the Mki set.

Step 7. Repeat steps 3 through 6 for each successive ramp until the end of the corridor is reached.

Step 8. If desired, the results may also be summarized in an origin-destination matrix table.

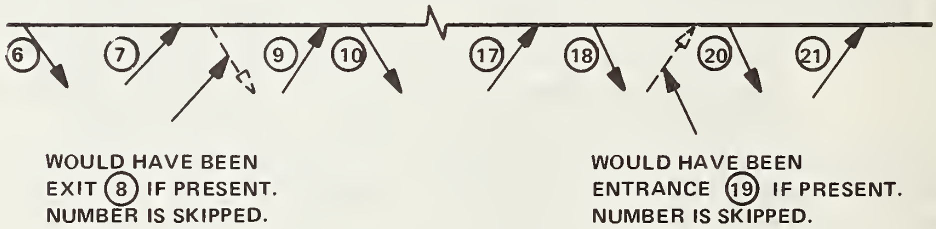
SAMPLE PROBLEM

A fictitious corridor whose main limited access facility is the Omega Freeway, contains the entrance and exit ramps listed sequentially in Table 56.

Step 1. The ramp numbers have been assigned as shown in Table 56, using odd numbers for entrance ramps and even numbers for exit ramps. Note that the first entrance ramp is the freeway input at the start of the corridor, and the last exit ramp is the freeway output at the end of the corridor. (The "distance" column need not be filled in at this time. It will be used later in the trip length determination.)

Step 2. A typical work sheet is shown (completed) in Figure 32. It begins with the line diagram running across the center of the sheet, showing the input data (balanced volume network). Add all ramp numbers along the line diagram, and entrance ramp numbers as shown at the left edge.

In the event that there are two (or more) consecutive entrance ramps (no intervening exit ramp), skip the exit ramp number that would have been used had the exit ramp been present. Follow a similar procedure for 2 consecutive exit ramps. This is illustrated in the sketch below.



Step 3. Calculate first exit fraction:

$$f2 = \frac{R2}{M1} = \frac{450}{3850} = .117$$

List exit fraction as shown in Figure 32.

Step 4. Calculate exit ramp components:

(This first case is a trivial one since there is only one component) $R21 = f_2 \times M_1 = (.117)(3850) = 450$

List R21 under exit ramp 2, opposite Origin 1, as shown in Figure 32.

Table 56. Listing of Ramps for Sample Problem

Entrance Ramps Name	Entrance Ramps Assigned No.	Exit Ramps Name	Exit Ramps Assigned No.	Distance From Corridor Start (MI)
Omega Fwy	1			
		Alpha St.	2	
Beta Ave.	3			
		Gamma Blvd.	4	
Delta Drive	5			
		Epsilon St.	6	
Zeta Ave.	7			
		Eta Ave.	8	
Theta St.	9			
		Iota Rd.	10	
Kappa Drive	11			
		Lambda Lane	12	
Mu Ave.	13			
		Nu St.	14	
Xi St.	15			
		Omicron Way	16	
Pi Place	17			
		Rho Blvd.	18	
Sigma St.	19			
		Omega Fwy	20	

Note: 1 mile = 1.609 km

Step 5. Calculate the mainline volume components downstream of ramp 2:

M21 = M1 - R21 = 3850 - 450 = 3400

M21 denotes, that of the original volume that entered at ramp 1 (the freeway mainline in this case), 3400 remains after exit ramp 2.

List M21 above M2, opposite origin 1, as shown in Figure 32.

Step 6. Add entrance ramp volume from entrance ramp 3 to mainline components:

M31 = 3400 as previously

$\underline{\text{M33}} = \underline{1200}$

$\sum$M3i = 3600 = total mainly volume = M3

List the M3i above M3, opposite their respective origins, as shown in Figure 32.

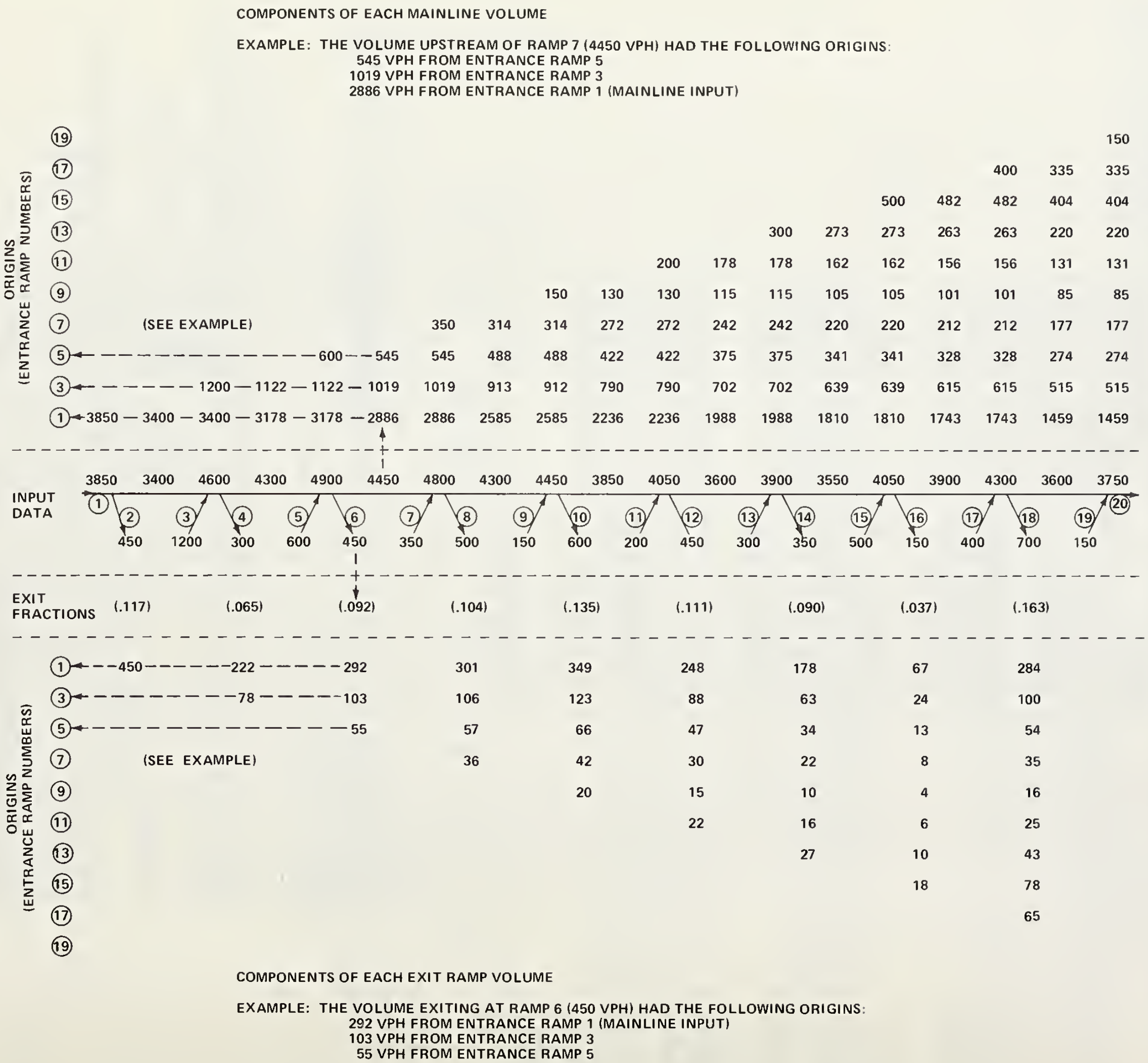


Figure 32. Typical Worksheet for Origin-Destination Pattern Estimation

Step 7. Repeat procedure, listing the results of each calculation as before:

a) $f4 = \frac{R4}{M3} = \frac{300}{3600} = .065$

b) $R41 = f4 \times M31 = (.065)\ (3400) = 222$
$R43 = f4 \times M33 = (.065)\ (1200) = 78$

Thus, of the 300 vph exiting at ramp 4, 222 vph came from ramp 1 (the mainline input) and 78 came from entrance ramp 3.

c) $M41 = M31 - R41 = 3400 - 222 = 3178$
$M43 = M33 - R43 = 1200 - 78 = 1122$

Thus, the components of the mainline volume at point 4 consist of 3178 vph from entrance ramp 1 and 1122 from entrance ramp 3.

d) Add entering volume to mainline:

$M51 = M41 = 3178$
$M53 = M43 = 1122$
$M55 = R5 = 600$

One additional cycle is calculated for demonstration:

e) $f6 = \frac{R6}{M5} = \frac{450}{4900} = .092$

f) $R61 = f6 \times M51 = (.092)\ (3178) = 292$
$R63 = f6 \times M53 = (.092)\ (1122) = 103$
$R65 = f6 \times M55 = (.092)\ (600) = 55$

g) $M61 = M51 - R61 = 3178 - 292 = 2886$
$M63 = M53 - R63 = 1122 - 103 = 1019$
$M65 = M55 - R65 = 600 - 55 = 545$

h) $M71 = M61 = 2886$
$M73 = M63 = 1019$
$M75 = M65 = 545$
$M77 = R7 = 350$

This process is continued until the end of the corridor is reached. The results for the total corridor are shown on the work sheet. Note that with the layout shown, the full origin-destination matrix is available on the work sheet. For example, at exit ramp 10, one can see that 123 vph exiting came from entrance ramp 3. Similarly, on the mainline just downstream of exit ramp 10, there are 790 vph remaining of the 1200 that entered at ramp 3.

Step 8. For illustration, the origin-destination matrix is recorded as shown in Table 57.

Table 57. Typical Format for Origin-Destination Matrix

Entrance Ramp		Exit Ramp: Alpha St 2	Gamma Blvd 4	Epsilon St 6	Eta Ave 8	Iota Rd 10	Lambda Lane 12	Nu St 14	Omicron Way 16	Rho Blvd 18	Omega Fwy 20	Total Entrance Ramp Volume
Omega Fwy	1	450	222	292	301	349	248	178	67	284	1459	3850
Beta Ave	3	-	78	103	106	123	88	63	24	100	515	1200
Delta Drive	5	-	-	55	57	66	47	34	13	54	274	600
Zeta Ave	7	-	-	-	36	42	30	22	8	35	177	350
Theta St	9	-	-	-	-	20	15	10	4	16	85	150
Kappa Drive	11	-	-	-	-	-	22	16	6	25	131	200
Mu Ave	13	-	-	-	-	-	-	27	10	43	220	300
Xi St	15	-	-	-	-	-	-	-	18	78	404	500
Pi Place	17	-	-	-	-	-	-	-	-	65	335	400
Sigma St	19	-	-	-	-	-	-	-	-	-	150	150
Total Exit Ramp Volume		450	300	450	500	600	450	350	150	700	3750	

APPENDIX B

TRIP LENGTH ESTIMATION

B.1 OBJECTIVE: To develop an estimate of median trip lengths* during a typical peak hour on the major limited access facility or facilities within the corridor. The results will be used subsequently to evaluate diversion potential and estimate benefit reduction associated with fewer diversion points.

B.2 INPUTS REQUIRED:
(1) Origin-destination worksheet
(2) Distance of each ramp from start of corridor. An accuracy of 0.1 mile (0.16 km) is adequate.

B.3 OUTPUT: A composite average value of median trip length for the corridor.

B.4 PROCEDURE:

Step 1. For the given freeway, enter the distances of each ramp from the start of the corridor on the ramp listing used previously for the origin-destination estimation.

Step 2. Prepare a table with the following headings: "Entrance Ramp Number," "Exit Ramp Number at 1/2 Volume", "Distance Between Entr. and Exit Ramp". (For example, see Table 59)

Step 3. Refer to the origin-destination worksheet. Corresponding to each origin (entrance ramp) shown as the upper left "scale" on the worksheet, read horizontally to the right until the volume has dropped to approximately one-half of its initial value. Note the exit ramp number at which this occurs and record this on the table next to the corresponding entrance ramp. Continue the process for each succeeding entrance ramp until the one-half volume points fall beyond the last exit ramp of the freeway.

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*Median trip length is defined as the distance travelled on the freeway by at least 50% of vehicles entering at a given point

Step 4. Using the data from Step 1, calculate the distance between each entrance ramp/exit ramp pair and record it on the table. These represent the individual median trip lengths associated with each entrance ramp.

Step 5. Repeat the previous steps for any additional freeways which run the length of the corridor.

Step 6. Compute the average of the individual median trip lengths for each freeway*. Then average the freeway values to obtain the composite average median trip length for the corridor.

B.5 SAMPLE PROBLEM: The data for the fictitious Omega Freeway (used for the origin-destination pattern estimation) are again used in this sample problem.

Step 1. The distance of each ramp from the start of the corridor have been tabulated on the previous listing of ramps (from the origin-destination estimation) as shown in Table 58.

Step 2. The table for entering the data is shown as Table 59. (The results of the subsequent steps have already been entered in the table).

Step 3. Refer to the origin-destination worksheet in Appendix A (Figure 32). Starting with entrance ramp 1, its initial volume is 3850. Reading horizontally to the right we look for the last point that the volume still exceeds one-half of its initial value (i.e., 1925). It is seen that this point is where the volume is 1988, (since the next volume is 1810). The corresponding exit ramp is ramp #14. Thus at least 50 percent of those entering at ramp 1 will travel as far as exit ramp 14. For entrance ramp 3 (initial volume of 1200) we look for the last point where the volume still exceeds 600. This occurs at exit ramp 18. Similarly for entrance ramps 5 and 7, the exit ramps for at least one-half volume remaining are numbers 18 and 20, respectively. For all remaining entrance

*Actually, a "weighted" average (based on entrance ramp volumes) could be used. However, several sample calculations have shown that the difference will not be significant. Therefore, the simple average is considered adequate.

Table 58. Listing of Ramps for Sample Problem

Entrance Ramps		Exit Ramps		Distance From Corridor Start (MI)
Name	Assigned No.	Name	Assigned No.	
Omega Fwy	1			0
		Alpha St.	2	1.1
Beta Ave.	3			1.4
		Gamma Blvd.	4	1.6
Delta Drive	5			1.9
		Epsilon St.	6	3.0
Zeta Ave.	7			3.2
		Eta Ave.	8	4.2
Theta St.	9			5.1
		Iota Rd.	10	6.0
Kappa Drive	11			6.2
		Lambda Lane	12	7.5
Mu Ave.	13			8.0
		Nu St.	14	9.0
Xi St.	15			9.3
		Omicron Way	16	10.5
Pi Place	17			10.8
		Rho Blvd.	18	12.0
Sigma St.	19			12.2
		Omega Fwy	20	14.0*

(Note: 1 mile = 1.609 km)

* to next exit

Table 59. Median Trip Length (MTL) Worksheet

Entrance Ramp No.	Exit Ramp No. At 1/2 Vol.	Dist. Between Entr. & Exit Ramp (=MTL)
1	14	9.0
3	18	10.6
5	18	10.1
7	20	10.8

(For all other entrance ramps, the 1/2 volume points fall beyond the end of the corridor)

$$\text{Composite Average MTL} = \frac{9.0 + 10.6 + 10.1 + 10.8}{4}$$

$$= 10.1 \text{ Miles (16.3 kilometers)}$$

ramps, the one-half volume points fall beyond the end of the corridor and are thus not included in the calculation. The exit ramps corresponding to each included entrance ramp have been entered in Table 59.

Step 4. Using the last column of Table 58, the distances between each entrance ramp/exit ramp pair have been calculated and entered in Table 59. For example, entrance ramp 5 is 1.9 miles (3.1 kilometers) from the start of the corridor, while exit ramp 18 is 12 miles (19.3 kilometers) from the start. Thus, the difference between these is the distance between entrance ramp 5 and exit ramp 18, i.e., 10.1 miles (16.3 kilometers).

Step 5. The present example includes only one freeway, and thus we proceed to step 6.

Step 6. The computation of the composite average median trip length is shown in Table 59. The result for this sample problem is 10.1 miles (16.3 kilometers).

APPENDIX C

PROCEDURE FOR COMPUTATION OF CONTROL PROBABILITY FACTORS

To develop a set of control probability factors, as given in Table 32 of the text, a probabilistic model is utilized. This model is based on multi-variate normal distribution concepts. The corridor data required to implement this model consists of sets of hourly volume count data. These sets record flow levels which exist on a peak hour basis over a period of 20 to 30 weekdays. Each set of data represents flow conditions existing at a single roadway location. For each data set, a site-specific mean volume and standard deviation is calculated. A corridor-wide mean and standard deviation is obtained by a weighted linear combination of the site-specific quantities. The weighting coefficients are the number of data points at a site divided by the total number of data points collected in the corridor. Standard equations for computing the mean and standard deviation data set are:

Mean:
$$\overline{Q}_j = \frac{1}{nj} \sum_{i=1}^{nj} Qij \tag{44}$$

Standard Deviation:
$$\sigma_{Q_j} = \sqrt{\frac{1}{nj} \sum_{i=1}^{nj} (Qij-\overline{Q}j)^2} \tag{45}$$

where:

- Qij - ith volume data point at jth site
- nj - number of data points at jth site
- $\overline{Q}j$ - mean volume level at jth site
- σ_{Q_j} - standard deviation at jth site

These two parameters completely define the density function for a normal probability distribution. The probability distribution is used to model the variations in the flow. The probability distribution thus provides a basis for describing the dynamic flow characteristcs which exist throughout the corridor. The concept of the model can be visualized by considering the specific roadway configuration given in Figure 33. The hourly variations in flow level (per lane) on each freeway section downstream of the direct connector is assumed as a normal probability distribution. The standard form for the density function of this distribution is:

$$f(Q) = \frac{1}{\sqrt{2\pi}\ \sigma_Q}\ e^{\left(\frac{-(Q - \overline{Q})^2}{2\sigma_Q^{\ 2}}\right)} \tag{46}$$

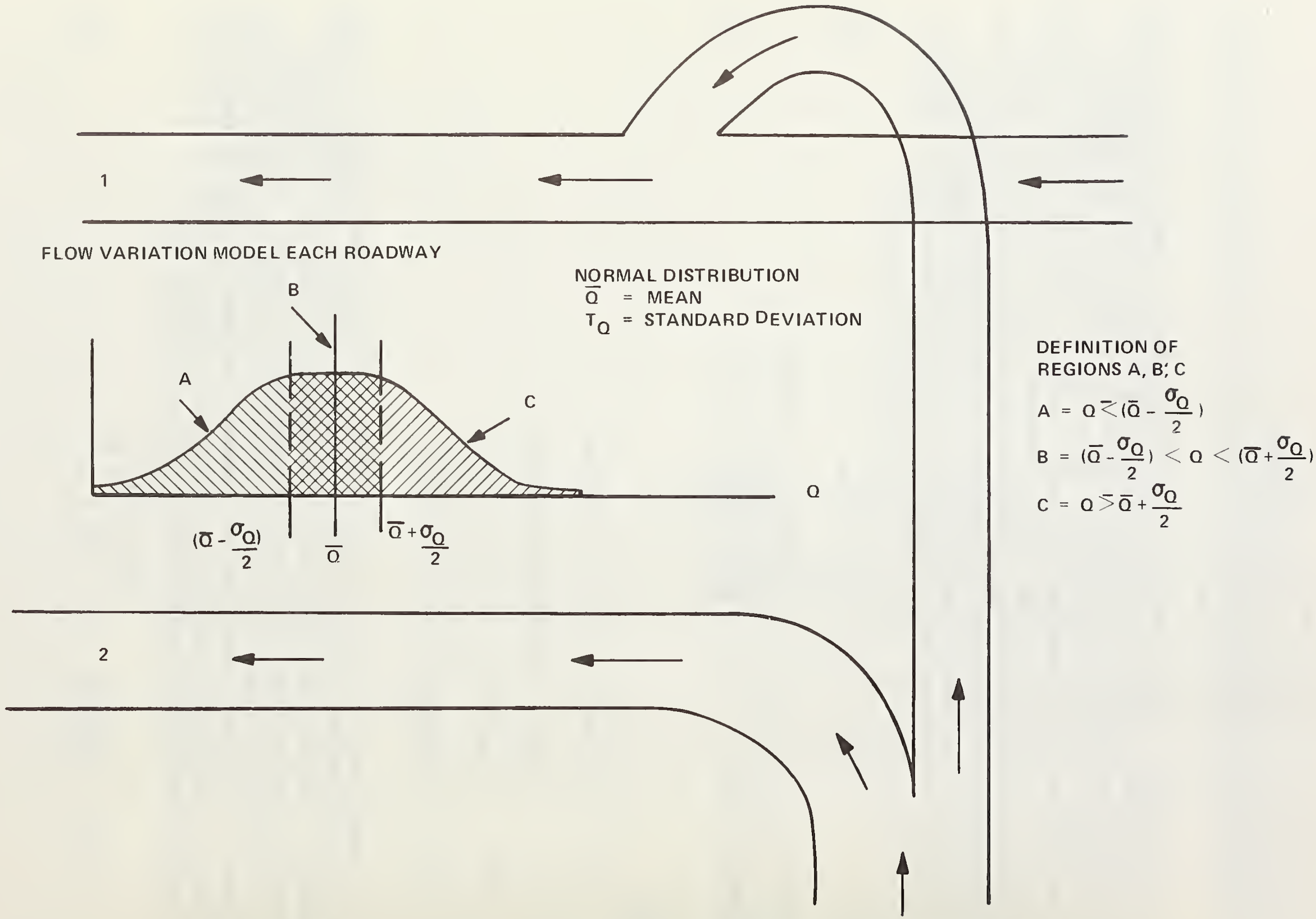

Figure 33. Roadway Diversion Configuration with Probabilistic Flow Model

Once the mean and standard deviation have been estimated using data available from specific corridor locations, the model itself, can be used to establish the variability of conditions throughout the corridor.

Returning to Figure 33 and using the properties of a normal distribution three flow regions are defined each with a probability of occurrence. The three flow regions are:

Region A. $Q \leq (\bar{Q} - \frac{\sigma Q}{2})$. Flow substantially below mean value - excess capacity available for control;

Region B. $(\bar{Q} - \frac{\sigma Q}{2}) < Q < (\bar{Q} + \frac{\sigma Q}{2})$. Flow within a range of mean value - excess capacity assumed not available for control;

Region C. $Q \geq (\bar{Q} + \frac{\sigma Q}{2})$. Flow sustantially above mean value - no excess capacity available.

The set of control probability factors (Table 32 of the text) were developed using the following probabilities of occurrence of each region, as defined by the normal probability distribution:

Pr (A-occurrence) = .3

Pr (B-occurrence) = .4

Pr (C-occurrence) = .3

At this point the variational characteristics of each roadway by itself has been described. The final step is to determine the joint variational characteristics of the roadways taken together as a network. These joint characteristics are determined with the assumption of independence of roadway flows. This assumption allows the joint flow probabilities to be obtained as the product of the individual roadway probabilities.

The exercise of corridor control, during each of the operational conditions listed in Table 32 of the text, can now be addressed.

The procedure for computing control probability factors is based on the probability of A, B and C occurences. The probability of occurrences is in turn based on the size of the A, B or C regions in relation to the shape of the density function. The shape of region B is centered about $\overline{Q}$, with the width set equal to the standard deviation, σ_Q.

The procedure is best illustrated by examining the calculation of the probabilities in Table 32 of the text. Referring to Figure 33, each of the nine table elements will be considered. First with regard to peak period normal conjestion, for a configuration of two freeways with "diversion only," control can be implemented only during the time periods when freeway 1 is operating in flow regions A or C

and at the same time freeway 2 is operating in flow regions C or A. The probability of occurrence of this combined condition is (.3) (.3) + (.3) (.3) equal to .18. Hence on a day to day basis, diversion only control between both freeways would occur with a probability of 0.18 or, on the average, 1.8 peak hours out of every 10 peak hours. The second operational scenario of a single freeway with alternate and ramp metering with signal control has a control probability factor of .3. This factor is based on the observation that for control to be exercised the freeway must operate in region C. When control is exercised the arterial signal control maintains arterial performance in the presence of the traffic flow shifted from the freeway. Thus each time conditions on the freeway warrant a shift of traffic the alternate is able to accommodate this shift with the probability of close to 1. The control probability is therefore (.3) (1). The operational scenario of two freeways with alternate and all control functions available has a control probability factor of .51. This probability is made up of three occurrence probabilities: the probability that freeway 1 is in region C, the probability that freeway 2 is in region C and the probability that both freeways are operating in region C simultaneously. These three probabilities are respectively .3, .3 and .09. The control probability factor is the union (probabilistic) of region C on freeway 1 with region C on freeway 2 or (.3 + .3) - .09 = .51.

For the same set of operational scenarios acting with a peak period incident condition, the corresponding control probability factors are .3, 1.0 and 1.0 respectively. The factor .3 is obtained for the two freeway diversion only scenario. The basis for a .3 factor is that given that a capacity-reducing incident has occurred on one freeway, the other freeway must be operating in region A for diversion control to be implemented. The 1.0 factor for the other scenarios reflects the policy that control will always be implemented to minimize the effects of the incident congestion.

When the capacity-reducing incident occurs during an off-peak period the corresponding control probability factor is 1.0 for all scenarios. The basis for this probability is that during an off-peak period there is always excess capacity available on an alternate roadway. Therefore when a capacity-reducing incident occurs during the off-peak period a shift of traffic can always be made.

The utilization of this probabilistic modeling approach provided a firm basis for the characterization of the variability of traffic. This approach thus defines the extent of the interaction between a real-time surveillance and control system and the corridor flow dynamics. The key to this approach is the determination of the variability of the flow on and between the roadways of the corridor. The standard deviation of the normal distribution is the parameter which quantities variability. The assembly of the data base is critical to developing this required estimate.

APPENDIX D

PROCEDURE FOR SELECTING CRITICAL INTERSECTIONS

D.1 INTRODUCTION

This appendix describes a procedure which may be used to decide which candidate intersections should be designated as CIC's. The first step in the procedure is to gather cycle-by-cycle volume counts at the candidate locations.

In the evaluation step, cycle volume and phase timing data at each candidate intersection are used to judge the value of CIC operation. The following criteria are considered:

- The number of cycles out of 25 cycles during peak conditions that would have required a split adjustment because of alternation of demand from one phase to the other.

- The average amount of green time required by the volume of each phase, compared to the amount of green time available.

A combination of heavy demand for split adjustments and moderate demand for green time is necessary to warrant CIC operation. Moderate green demand on both approach directions is necessary because there is no unused green time to borrow if both approach directions are saturated.

D.2 DATA ACQUISITION

Cycle-by-cycle discharge volume counts and green indication lengths are to be collected on the critical approaches* over 25 successive cycles during the highest volume rush hour period. The cycle-by-cycle volume counts are then converted to equivalent "per cycle per lane" counts and plotted. Typical plots are shown in Figures 34 and 35. Figure 34 shows the number of vehicles per lane discharged for each of the 25 cycles, for the two conflicting critical (heaviest volume) approaches at a potential CIC intersection. Figure 35 shows the same data for an intersection which would not warrant CIC treatment. The tabulation in the upper right-hand corner gives pertinent information extracted from the plots for evaluation purposes.

The data of most significance is the number of instances in which, for successive cycles, the volume change on each approach is greater than or equal to 3 vehicles per lane. A volume change of this magnitude is sufficient to call for a nominal change in split (3 veh. x 2.1 sec/veh = 6.3 sec to be added to one phase and subtracted from another). Thus, we look at volume changes from cycle to cycle in place of calculation of the required split for each cycle.

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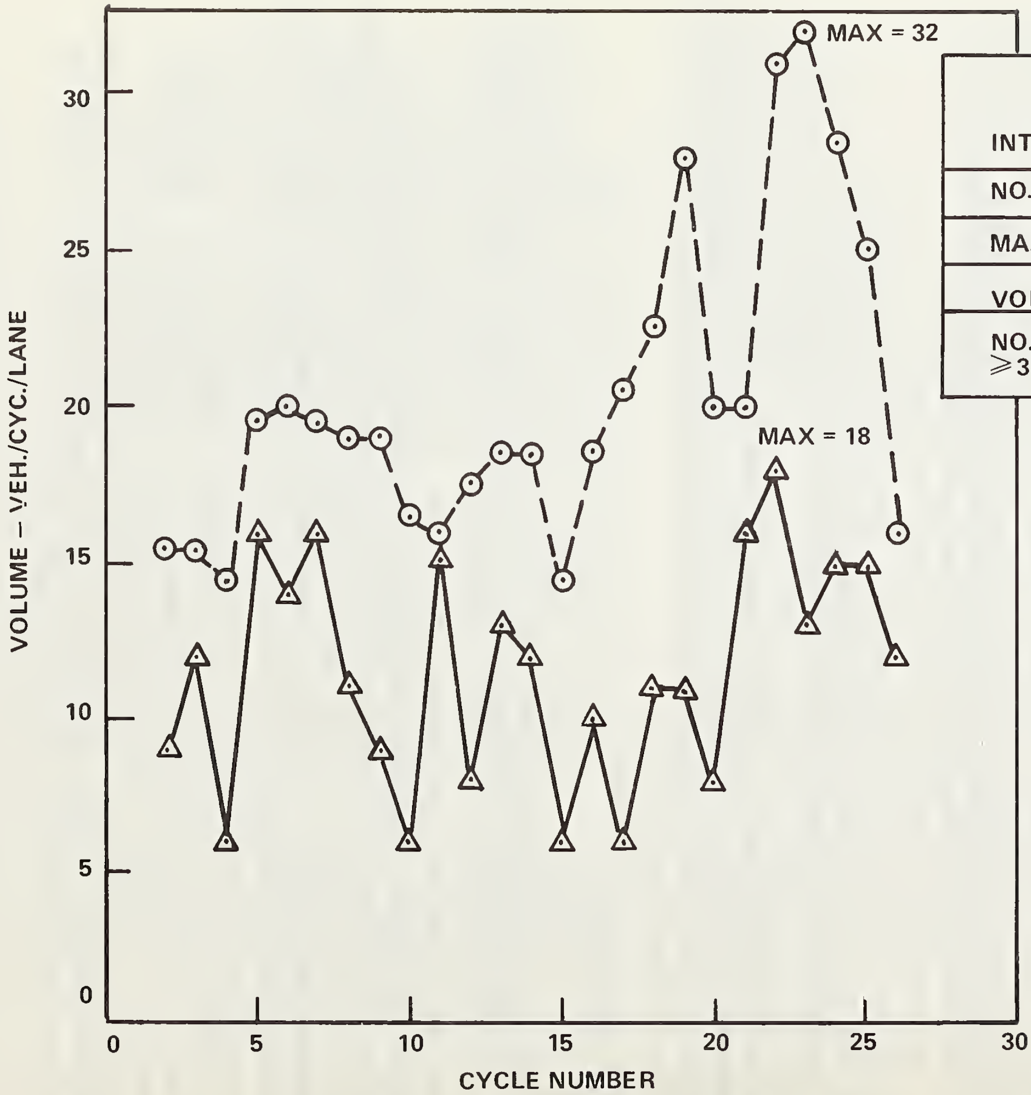
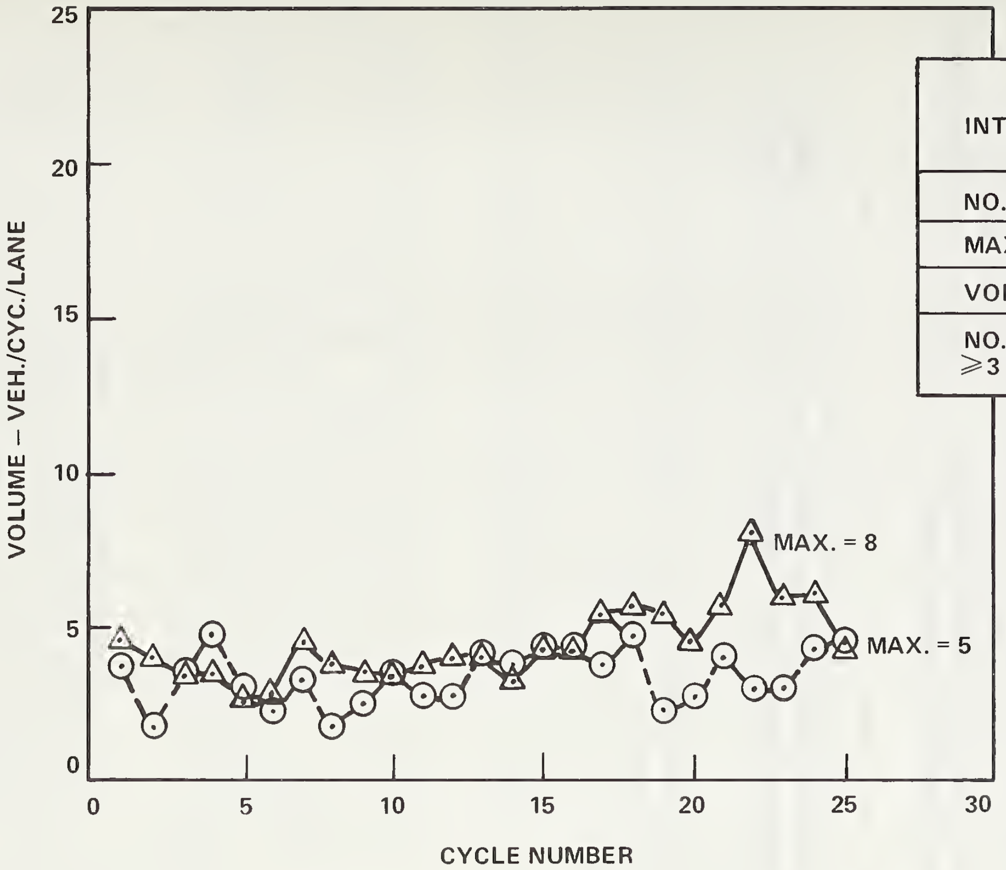
* If the approaches with the highest critical volumes cannot be identified before the data are taken, counts should be made on each approach.



INTERSECTION	△ 27TH	⊙ VINE
NO. LANES	3	2
MAX. VOL.	18	32
VOL. RANGE	12	17.5
NO. CHANGES ⩾3 VEH	16	10

Figure 34. CIC Evaluation Plot (27th and Vine)



INTERSECTION	△ 13TH	⊙ M
NO. LANES	4	4
MAX. VOL	8	5
VOL. RANGE	5.5	3
NO. CHANGES ⩾3 VEH.	0	0

Figure 35. CIC Evaluation Plot (13th and M)

D.3 WARRANTS FOR CIC OPERATION

The indices used to evaluate CIC operation are:

- Percent of green capacity utilized
- Number of changes $\geq$ 3 vehicles

The latter value is obtained directly from the graphs made for each intersection, such as Figures 34 and 35. The percent of green time utilized for each phase is obtained from the formula:

$$\text{percent green time used} = \frac{\text{Average Volume x 2.1 sec/veh}}{\text{Green time}} \tag{47}$$

where 2.1 sec. is the average vehicle headway. Available green time and average volume are obtained from the acquired data.

The "Warrant" used for CIC operation has two conditions:

- The data must snow no less than 9 cycles out of 25 in which the volume change from the previous cycles was at least 3 vehicles
- The green time used must be between 40 and 75 percent on both approaches.

APPENDIX E

CODED MESSAGE VERSUS VOICE COMMUNICATIONS
(Extracted From Report No. FHWA-RD-IP-76-11, "Motorist Aid System-State of the Art," Sept. 1976)

The question of voice versus coded communications has long been a controversial issue. The intuitive or generally perceived advantages of voice versus coded systems have been argued in several earlier references (Ref. 30, 72). Position papers from AASHTO's Subcommittee on Communications and Electronic Applications for Highways has taken a fairly firm stand in favor of "two-way voice duplex communication." Their original posture has been relaxed somewhat in that the most recent publication recognizes that there may be factors such as economics or some other specific local consideration that could justify the use of coded message transmission (Ref. 7).

Review of the operating experience confirms that many police agencies have a decided preference for voice communication, since it is consistent and compatible with the normal mode of police communications. This is an important consideration where the police agency is to be responsible for the dispatching function for motorist aid. In the case of the Northway system in New York, the police dictated that the system would be a voice system, and expressed an intention to refuse to participate should a coded message system be selected. When, for economic reasons, it was decided to install a coded system on I-45 in Houston against the wishes of the City Police Department, the system was abandoned after 22 months of operation on the advice of the police. After expressing a desire for a voice system, the Illinois Police refused to be involved as the operating agency when a coded system was selected for I-55; hence, the DOT had to provide their own dispatch facility and staff.

While operating procedures of coded message systems vary, there are a number of projects that dispatch a patrol vehicle initially regardless of which button is actuated (i.e., aid requested). Other systems may dispatch a patrol vehicle for mechanical service request or for medical or fire (or some combination thereof). While it may be argued that this type of response procedure alleviates the problem of gone-on-arrivals, current FHWA policy specifically requires that ". . . (aid requests) are served by responses which correspond to the coded messages sent from the sending location . . ."* There remains a pronounced difference of opinions among the projects on the effectiveness of responding directly to the aid request.

The two-way voice capability inherently provides a better definition of motorist need by direct conversation with the involved motorist or an observer at the site. Accordingly, voice systems do not experience the same problem as coded systems since it is seldom necessary to dispatch an initial vehicle to verify or determine the need. In the case of serious, possibly multi-vehicle accidents, the number and type of aid vehicles may not be immediately obvious to the dispatcher even with direct

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* Paragraph 5.a(1) of Vol. 6, Ch. 8, Sec. 3, Subsec. 3 of the Federal Aid Highway Program Manual.

verbal contact, but patrol cars on-site are required in such cases and can make necessary judgments as to whether additional equipment is needed.

While the advantages of voice systems in terms of response procedures are beyond question, a common disadvantage was cited by the systems studied; namely, the inordinate amount of dispatcher time expended on motorist aid calls. Under the best circumstances, more time is required to converse with the user than is required to monitor a coded control console.

Voice systems in ethnically-mixed sections of the country are also besieged by the need for bilingual dispatch operators. Although this is cited as a drawback, it stands to reason that if bilingual dispatchers are desirable for motorist aid functions, they are most surely needed as well for police communications with the public. Even operating staffs who expressed general satisfaction with their voice systems indicated an awareness of certain advantages in a coded message system. Particularly where police dispatchers are overburdened with police communications, lengthy conversations to specifically identify the problem or to give directions or weather conditions make some voice systems less attractive to the operating personnel.

Inasmuch as the operating agency is usually a law enforcement organization, the typical dispatcher is primarily responsible for police communications activities and must accommodate motorist aid calls within this framework. Moreover, a number of motorists will use the familiar telephone handset to obtain directions, roadway conditions, other types of information, or make inappropriate time-consuming requests such as relaying messages to a third party. While some systems accept these responsibilities as part of the level of service intended, most operating staffs resent the time spent in answering and advising the callers that such requests are beyond the scope of services offered.

From the motorist's standpoint, there is little doubt that voice communications can provide more assistance and service by having the ability, if requested, to supply information, relay messages, or transfer the call to someone who can dispatch the requested aid directly. Such flexibility is a decided advantage of voice systems, particularly where the intent is to provide a full scope of motorist services.

It would appear that with voice communications, by giving the motorist the assurance that aid is on its way and by answering any anxiety about cost or payment, the incident of gone on arrivals would be significantly decreased. Unfortunately, GOA data for both types of systems are sparse and fail to clearly support this assumption.

Experience with the Florida I-7 coded message system indicated that, even without voice communication, it becomes possible after a time to recognize patterns of calls which signify certain types of needs or severity of needs. Both the operating agency and the State DOT have expressed a high level of satisfaction with the coded system and are currently in the process of expanding it.

As shown in Table 10, there are identifiable advantages and disadvantages associated with each of these two operations. The factors listed are associated solely with the functional aspects of the detection, definition, and dispatch elements and do not include such critical concerns as capital cost or maintenance requirements. The

ultimate decision must, therefore, be based on a tradeoff analysis among the perceived needs and systems objectives, operational convenience, facility characteristics, and finally, system cost.

Table 10. Comparative Benefits of Response Procedures by System Type

CODED SYSTEM

ADVANTAGES	DISADVANTAGES
• Consumes less dispatcher time • If patrol vehicle initially dispatched, experienced patrolman can: - Better evaluate needs - Control and safeguard site - Guard against secondary incident - Keep traffic flowing - Administer first aid or rapidly obtain medical advice via police radio • No need for multilingual dispatcher capabilities	• Imprecise definition of problem or need • Pushbutton equipment and operation not as familiar to users as telephone handset • If patrol vehicle initially dispatched: - May increase response time of ultimate aid vehicle - Distracts patrol vehicle from primary law enformcement responsibility - Increases patrol resource requirements (equipment and staff) • Easier to disregard incoming call

VOICE SYSTEM

ADVANTAGES	DISADVANTAGES
• Permits full definition of user need • Familiarity of user with telephone handset • Provides opportunity to relieve motorist anxiety through verbal assurance assistance is forthcoming • Can advise user what actions to take, if any, before aid arrives • Can provide information of cost of service and acceptable forms of payment	• Time-consuming for dispatcher • Distracts from primary police responsibilities • Some callers may have difficulty in explaining their needs • Difficulties in understanding (language, road noise, or technical problems) • Many users waste system resources by requesting services or information beyond the intended level of service

APPENDIX F

PROCEDURE FOR SELECTING RAMPS FOR METERING*

F.1 INTRODUCTION

In general, metering of a particular ramp will be desirable if mainline congestion often exists in the ramp vicinity, if ramp-related accidents are frequent, or if the corridor control system will require that it be metered. Section F.2 provides three appropriate guidelines to address these situations, and any one of them may be used to select a ramp as a candidate for metering.

Once a candidate has been identified, it is necessary to evaluate the capability of the ramp and adjoining roadways to improve conditions through metering. Section F.3 provides 5 criteria for this purpose. These criteria are used to determine whether the ramp demand is within practical limits, whether the ramp storage and alternate diversion capability is adequate, and whether the ramp geometry permits safe metering. Candidates are retained only if all five criteria are satisfied.

For general application, data to be used for comparing with the numerical values listed in the guidelines should preferably be gathered on site, but may be estimated using other sources of information. Where the characteristic of a particular ramp are not covered by the guidelines, engineering judgment should be used in making the metering decision. All data should be adjusted to reflect design-year values, and truck/bus fractions of volume should be converted to equivalent passenger car units (PCU's).

F.2 GUIDELINES FOR SELECTING CANDIDATE RAMPS FOR METERING

The three guidelines provided below are used to establish whether a given ramp should be selected as a candidate for metering. As noted above, if any one of the guidelines is satisfied, the ramp should be selected as a candidate.

Guideline 1: Level of Service

A ramp is a candidate for metering if the mainline link affected by ramp vehicles experiences Level of Service D or worse for thirty minutes a day during most weekdays, or for two hours a day during at least 50 days of each year. The mainline link to be used for this purpose is defined as the section of roadway between a point approximately three-fourths the distance to the next upstream on-ramp, and one-fourth the distance to the next downstream on-ramp. The level of service may be computed using measured or derived volumes and speeds in the procedures given in Chapters 8 and 9 of the Highway Capacity Manual, Highway Research Board Special Report 87, 1965.

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*Many of the concepts and techniques included in this appendix are based on information contained in NCHRP Project 3-22 Report "Guidelines for Design and Operation of Ramp Control Systems."

Guideline 2: Accidents

A ramp is a candidate for metering if the number of accidents in the ramp vicinity which are attributable to merging activities exceeds the average rate for all other ramps on the freeway by a factor of at least 2, and if the accident conditions can be alleviated by metering. The ramp vicinity for this guideline is defined as the mainline section spanned by points 1500 feet (457 meters) upstream and 1500 feet (457 meters) downstream of the ends of the acceleration lane. Typically, unsafe conditions caused by multi-vehicle merging, poor sight distances, steep grades, short acceleration lanes, or difficult weaving conditions can be helped. Accidents on both the mainline and on the acceleration lane are to be included in this count.

Metering installed solely for the purpose of reducing accident rates need not be centrally controlled.

Guideline 3: System Integration

A ramp is a candidate for metering if required for integration with the corridor control system, even though it does not meet the requirements of either Guideline 1 or Guideline 2. This system guideline permits more precise access control to be exercised where necessary, and also provides control for those otherwise uncontrolled ramps whose volumes might be significantly increased as a result of vehicle diversion.

F.3 CRITERIA FOR DETERMINING SUITABILITY OF RAMPS FOR METERING

The five criteria provided below are used to determine whether the traffic and geometric characteristics associated with a given ramp are amenable to ramp metering. All five criteria must be satisfied; otherwise the candidate ramp is considered unsuitable and should be rejected.

Criterion A: Minimum Ramp Approach Demand

A ramp is suitable for metering if ramp volume is above the lowest metering rate acceptable to most motorists (about 240 PCU/hour or 4 PCU/minute) and if the ramp and/or alternate routes permit a reasonable amount of metering to be imposed. If demand exceeds the metering rate for a long period of time (i.e., to the extent that the ramp queue interferes with surface street operations), a suitable alternate route must be available to accommodate the excess vehicles, while if excess demand is characteristically of a short term nature then it may be adequately handled by storage on the ramp and local streets (Criterion B).

When an alternate route is available, the value of minimum demand chosen will depend on the estimated improvement to be obtained by denying access to a small number of vehicles per hour as well as the smoothing effect of metering. A minimum ramp demand value of 240 + 50n PCU/hour (where n is the number of mainline lanes) is recommended as a guide.

Criterion B: Non-diversion Metering

When no alternate route is available, metering provides only the smoothing of short-term peaks, which is useful under some circumstances. The factors influencing the selection of metering for this case include the time-varying characteristics of demand as well as the ramp and local street storage capability. The relationship among these parameters may be represented by the expression:

$$V_D - V_M = NL/st$$

where:

V_D = ramp demand in PCU/minute

V_M = ramp metering rate in PCU/minute

N = no. of ramp and local street lanes available for storage

L = ramp and local street vehicle storage length, in feet measured upstream of probable stop bar location (1 foot = .3 meter)

s = effective stored vehicle length (typically 25 feet (7.6 meters))

t = time, in minutes, measured from arrival of first vehicle of demand group entering at a fixed rate

Criterion C: Maximum Ramp Approach Demand

A ramp is suitable for metering if maximum projected ramp demand is less than the rate than can readily be metered one vehicle at a time. Maximum metering rate is a function of ramp geometry, grade, and merge region capacity. The maximum metering rate recommended for a well-designed ramp is 900 vehicles per hour where the merge area consists of a conventional single-lane acceleration section. If the demand exceeds this maximum and there is no viable alternate route for the excess, the ramp cannot be metered.

Criterion D: Ramp Storage

A ramp is suitable for metering if ramp storage space, in conjunction with available alternate routes, is sufficient to avoid queueing that interferes with local traffic near the ramp entrance. Ramp length required for vehicle storage may be estimated using the following expression:

$$\text{Storage length required (feet)} = L = \frac{5V_M(1.5\,T_A - T_F)}{12N}$$

(note: 1 foot = .3 meters)

where:

V_M = metering rate in PCU/hour

T_A = estimated alternate route travel time in minutes

T_F = estimated freeway travel time in minutes (not including ramp delay)

N = number of lanes

This expression is based on four assumptions: (1) each vehicle on the ramp queue requires 25 feet (7.6 meters) of storage length, (2) at equilibrium, all of the vehicles will divert when estimated alternate route travel time equals two-thirds the sum of ramp delay plus freeway travel time, (3) the alternate route can handle the difference between ramp demand volume and metering volume, and (4) demand exceeds metering rate.

The expression may be used to find what ramp length is required by using the estimated or measured values of travel times and desired metering rate. If the length is acceptable, then metering may be permitted. For example, if travel time difference ($1.5\ T_A - T_F$) is four minutes, and desired maximum metering rate is 500 VPH, the required two-lane ramp length is 417 feet (127.1 meters).

Alternatively, the expression may be used to find the allowable metering rate for a given ramp length.

Criterion E: Ramp Geometry

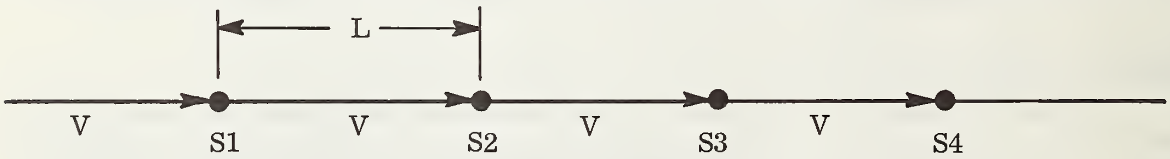
A ramp is suitable for metering if the ramp geometry provides safe stopping conditions approaching the ramp queue and safe merging conditions after vehicle release. For safe stopping, the ramp must provide the minimum stopping sight distances to each point upstream of the ramp stop bar that could become the end of a vehicle queue. For safe merging, the sight angles and distances to the lane 1 gap-decision region from all points downstream of the stop bar must be such that vehicles starting from the stop bar can merge at least as safely as they could prior to metering.

APPENDIX G

EFFECTIVENESS OF SIGNING STATIONS

The derivation of a relationship to quantify the relative effectiveness (i.e., the relative ability to shift a given traffic volume) between several sets of diversion subsystem designs is presented in this appendix. The principle design parameter which impacts sign complement effectiveness is the uni-directional roadway spacing of the diversion signing stations, each of which implies a diversion opportunity. Intuitively, it is clear that as the distance between diversion stations increases, or alternatively, as the number of stations over the corridor network decreases, the ability of the remaining complement of stations to shift flows between the roadways is reduced. The following approach provides a quantitative basis for estimating diversion effectiveness.

Consider an arbitrary section of freeway which has a constant traffic volume, V, (reasonably representative of a limited access freeway during peak hours) and where L is the distance between diversion sign stations. The stations are numbered starting at the upstream end of the roadway. (See sketch below).



In the model, it is assumed that motorists who will divert will do so at the first sign they come to (or not at all). Thus, at a sign location S2, the candidate divertees are motorists who have entered the freeway downstream of sign S1. Since the volume is constant, the number of candidate divertees is equal to the number who exit between S1 and S2.

The amount of the original volume remaining at any point (V_R) is representable by an exponential decay function of the form

$$V_R = Ve^{-\lambda L} \tag{50}$$

where λ is a constant which represents the exiting rate per unit distance. The number of candidate divertees (V_M) is then

$$V_M = V - Ve^{-\lambda L} \tag{51}$$

The actual number of divertees is a fraction of this. One distance-related factor which is a component of this fraction is how "relevant" the information may appear to be. For example, an incident or congestion a long distance away may be perceived to be of little relevance at this time because, even if the motorist will later go through that area, the time period may be perceived to be long

enough so that the congestion will not affect him, at least at the moment. This is just a mainfestation of the "future discount" phenomenon often encountered in human factors. As the sign spacing is increased, each sign must necessarily give information concerning a larger downstream section. As this section becomes larger, motorists will have a greater propensity to discount it.

With these factors in mind, the number of divertees approaching each sign is considered to be representable by:

$$V_D = V_M \, e^{-L/D} \tag{52}$$

where D is the "relevancy distance". (A typical value is about 12 miles (19.32 km)). Thus,

$$V_D = V\,(1 - e^{-\lambda L})\, e^{-L/D} \tag{53}$$

If the highway maintains its constant character, for n diversion points, the total number of divertees is:

$$\sum_n V_D = n\ V \left(1 - e^{\frac{-\lambda C}{n}}\right)\left(e^{-\frac{C}{nD}}\right) \tag{54}$$

where C is the length of the corridor.

If a system of n_1 diversion points is considered as the baseline (most versatile) system, the ratio of the number of divertees for smaller systems (with n_2 signs) is then:

$$R = \frac{n_2}{n_1} \frac{\left(1 - e^{-\frac{\lambda C}{n_2}}\right) e^{-\frac{C}{n_2 D}}}{\left(1 - e^{-\frac{\lambda C}{n_1}}\right) e^{-\frac{C}{n_1 D}}} \tag{55}$$

where

$$\lambda = \frac{1}{L} \ln\left[\frac{1}{\left(1 - \frac{Vm}{V}\right)}\right] \qquad \text{(from equation 51)} \tag{56}$$

Since λ is a constant, its value may be established from any one known set of conditions. One such set is obtained by recognizing that for a length (L) equal to the median trip length (MTL), the value of $Vm/V = 0.5$.
Thus,

$$\lambda = \frac{1}{MTL} \ln 2 \tag{57}$$

or

$$\lambda = \frac{.693}{MTL} \tag{58}$$

The following example illustrates the computation. Assume that for a traffic corridor 40 miles (64.4 km) long, a full complement baseline system contains 20 diversion signing stations (n_1) per limited access roadway direction. Assume further that the median trip length (MTL) has been calculated to be 6 miles (9.7 km). Then:

$$\lambda = \frac{.693}{6} = .115$$

For a reduced complement system design, say 15 sign stations per limited access roadway direction, the corresponding reduced effectiveness is:

$$R = \frac{15}{20} \frac{\left(1 - e^{-.115\left(\frac{40}{15}\right)}\right) \quad e^{\frac{-40}{15(12)}}}{\left(1 - e^{-.115\left(\frac{40}{20}\right)}\right) \quad e^{\frac{-40}{20(12)}}}$$

$$R = .914$$

Thus a numerical reduction of 25 percent in sign stations corresponds to a 8.6 percent reduction in relative diversion effectivenss for this example.

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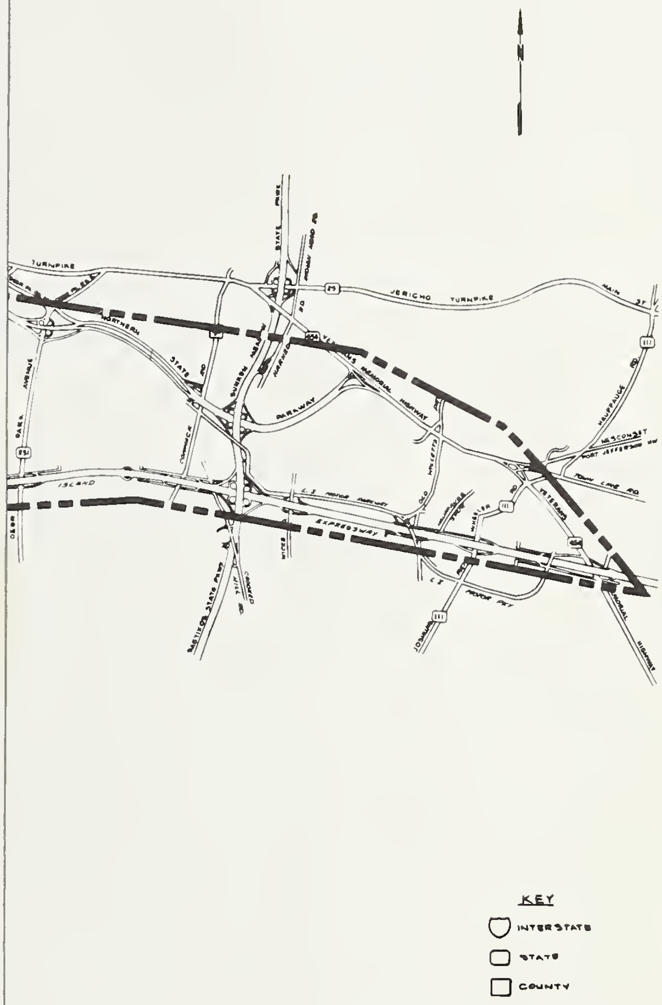

Figure 14. Example of Partitioning a
Corridor Into Control Subnetworks

Figure 14. Example of Partitioning a Corridor Into Control Subnetworks

FEDERALLY COORDINATED PROGRAM OF HIGHWAY RESEARCH AND DEVELOPMENT (FCP)

The Offices of Research and Development of the Federal Highway Administration are responsible for a broad program of research with resources including its own staff, contract programs, and a Federal-Aid program which is conducted by or through the State highway departments and which also finances the National Cooperative Highway Research Program managed by the Transportation Research Board. The Federally Coordinated Program of Highway Research and Development (FCP) is a carefully selected group of projects aimed at urgent, national problems, which concentrates these resources on these problems to obtain timely solutions. Virtually all of the available funds and staff resources are a part of the FCP, together with as much of the Federal-aid research funds of the States and the NCHRP resources as the States agree to devote to these projects.*

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems connected with the responsibilities of the Federal Highway Administration under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by keeping the demand-capacity relationship in better balance through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

* The complete 7-volume official statement of the FCP is available from the National Technical Information Service (NTIS), Springfield, Virginia 22161 (Order No. PB 242057, price $45 postpaid). Single copies of the introductory volume are obtainable without charge from Program Analysis (HRD-2), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements which affect the quality of the human environment. The ultimate goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge of materials properties and technology to fully utilize available naturally occurring materials, to develop extender or substitute materials for materials in short supply, and to devise procedures for converting industrial and other wastes into useful highway products. These activities are all directed toward the common goals of lowering the cost of highway construction and extending the period of maintenance-free operation.

5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural designs, fabrication processes, and construction techniques, to provide safe, efficient highways at reasonable cost.

6. Prototype Development and Implementation of Research

This category is concerned with developing and transferring research and technology into practice, or, as it has been commonly identified, "technology transfer."

7. Improved Technology for Highway Maintenance

Maintenance R&D objectives include the development and application of new technology to improve management, to augment the utilization of resources, and to increase operational efficiency and safety in the maintenance of highway facilities.

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